Summary of Findings

Cost Analysis of Depressing I-10 Through the Rio Nuevo Project Site

Submitted to

City of Tucson Department of Transportation

By

Johnson-Brittain & Associates

January 4, 2002

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10890
Michael Thomas
Johnson
DATE SIGNED 2/4/02
Arizona U.S.A.

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CITY OF TUCSON
Department of Transportation Engineering Division
P.O. Box 27210
Tucson, Arizona 85726-7210

RE: Cost Analysis of Depressing I-10
Through the Rio Nuevo Project Site
City of Tucson Job No. 1051
Johnson-Brittain No. 3303.01

Gentlemen:

We have completed the preliminary cost analysis of depressing I-10 though the proposed Rio Nuevo project site. We estimate that depressing the roadway will increase the cost $51 million above what ADOT would spend in completing the same section as currently planned. We recommend that $55 million be used for budgeting.

In determining the cost estimate, a number of issues have been examined and approaches developed. This report documents all of this work. The preliminary roadway geometrics that have been developed as part of this effort are documented in the accompanying set of plans.

We hope that this report provides the information you need to decide whether or not to pursue the depression option. If you have any questions regarding this report or the accompanying plans, please fell free to contact this office.

Sincerely,

JOHNSON-BRITTAIN & ASSOCIATES, INC.

Michael T. Johnson, P.E., R.L.S.
President

MTJ/rlq

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EXECUTIVE SUMMARY

Purpose of the Study. Rio Nuevo, a wide-ranging effort by the City of Tucson to revitalize the area south and west of downtown, is transected by I-10. The freeway is elevated through this area. Current plans by ADOT to widen I-10 would increase its height even more. The physical and psychological barrier created by I-10 would be mitigated to to a great some extent if the freeway were depressed. Visual quality would be markedly improved and noise would be reduced. All of this would work to enhance the success of the revitalization effort.

The value of depressing I-10 does not, however, extend to transportation benefits. Greater levels of capacity or safety would not be achieved beyond those of the elevated freeway. Accordingly, ADOT will not participate in the cost beyond what would be incurred under the current plan. It will be necessary for the City of Tucson to obtain funding for the increased cost of depressing the freeway.

The purpose of this study is to determine what that increase in cost would be. In doing so, it is necessary to develop approaches to a number of technical issues relating to roadway geometrics, drainage, infiltration, and so forth. This report documents the results of this effort.

Extent of Depression. The north end of the proposed depression would begin south of St. Mary's Road and extend southward about 1.3 miles ~~to approx~~[illegible]imately 22nd Street. The geometrics developed here tie into the future (elevated) mainline profile currently planned by ADOT at each end of the proposed depression.

Roadway Geometrics. Much of the effort here has focused on developing geometrics for the depressed mainline, ramps, frontage roads and crossings. The results are presented in the accompanying plans which would serve as the alternative to the general plan and to the mainline geometrics developed with the frontage road design should the proposal to depress I-10 be adopted. As with the elevated freeway, it is difficult to take up the elevation differential between the mainline and the frontage roads with the relatively short ramp lengths that are available. It is in some cases necessary to depress the frontage roads where before it was necessary to elevate them to make the ramp connections. For this reason, it will be necessary to reconstruct some portions of the recently completed frontage roads. Some existing retaining and noise wall will no longer be necessary.

Crossings. Bridges crossing the depressed freeway would be constructed at Congress Street, Clark Street , and Simpson Street/Mission Lane. A fourth structure, a pedestrian/bikeway, would be constructed between Clark and Simpson. The pedestrian/bikeway would span the frontage roads as well as the ramps and mainline. The other crossings would serve vehicular as well as pedestrian and bicycle traffic and would intersect with the frontage roads.

Future Deck Park. Because the pedestrian/bikeway bridge spans the frontage roads and the Clark Street Bridge does not, the geometrics proposed here, as presently formulated, would not allow a future deck park to be constructed between those structures. It would be necessary for the Clark Street structure to span the frontage roads as well. The viability of that has not been investigated here.

Pavement Drainage. The low point in the mainline profile is well below the hydraulic grade line of the adjacent Santa Cruz River, precluding directly draining the roadway by gravity. Three approaches for dealing with this have been investigated--(1) pumping the runoff directly to the river with a large pumping system sized to handle the 100-year peak discharge, (2) constructing an outfall storm drain downstream past Grant Road which would allow the sump to drain by gravity, and (3) storing the entire 100-year storm runoff in a concrete vault under the roadway to be pumped slowly with a smaller pumping system. The third option was found to be most cost-effective and has been recommended here.

Offsite Drainage. Inverted siphons have been proposed to carry offsite drainages under the depression. Residual water in the inverts after the storm would be pumped by a small pumping system. That system would be separate from the pavement drainage system to avoid the potential for offsite drainage backing up into the freeway.

Utilities. A number of utilities cross the proposed depression and will need to be lowered. Utilities adjacent to the frontage roads where they are to be depressed may need to be lowered as well. Sanitary sewers may need to be realigned.

Groundwater Infiltration. A strategy to prevent groundwater infiltration from the Santa Cruz River causing upward hydrostatic pressure on the roadway pavement and behind retaining walls has been developed by URS Corporation. The recommended approach is to construct drains from geocomposite material under the roadway and behind the retaining walls to drain off any infiltration that might reach the depressed roadway. This course of action would result in an additional 6.5 cfs or less being discharged into the pavement drainage system.

Potential for Contamination of Infiltration. URS also considered the possibility of infiltration becoming contaminated by underground pollutants as it travels from the Santa Cruz River to the depression. Were that the case, some treatment of the infiltrate might be required prior to it being discharged into the Santa Cruz River. A review of data from monitoring wells in the area suggests that this would not be the case, but more analysis including drilling is needed.

Retaining Walls. The height and location of retaining walls were identified during the development of the roadway geometrics. Approximately 12,200 linear feet of retaining walls are anticipated, ranging in height from 13' to 31'. URS investigated several approaches for constructing these walls and recommends that a soil nailing system be used.

Construction Sequencing. A plausible construction sequencing approach consisting of three main phases is presented in the report and in the accompanying plans. The primary concern is the closure of Congress Street across I-10 which would occur with the elevated roadway proposal as well. A second important concern is how to move 1.7 million cubic yards of excavation, 200,000 square yards of existing pavement, existing underpasses, and other material to be demolished and removed across mainline traffic being detoured along the frontage roads. The feasibility of alternate routes for removal of this material, such as temporary tunnels under the frontage roads, should be investigated during final design.

Cost. The proposal to depress I-10 though Rio Nuevo appears to be technically viable. The increase in cost is determined from this study is $51 million which includes design, construction administration and 15% contingency. It is recommended here that $55 million be used for budgetary purposes.

OVERVIEW

The Arizona Department of Transportation (ADOT) began improving I-10 through the Tucson area in 1990. That work is being accomplished in general accordance with the *I-10 General Plan, Ruthrauff Road to I-19* completed by JHK & Associates in January 1990 and includes new frontage roads as well as widening the mainline. The frontage roads are being constructed first so that they can be used to detour freeway traffic while the mainline is under construction.

The frontage roads have, at this point, been designed and for the most part constructed. Design of the first of three sections of mainline widening, *I-10, Prince Road to Grant Road*, is currently under design and the second, *I-10,Grant Road to St. Mary's Road* will be started in early 2002. The final section, *I-10, St. Mary's Road to I-19* is expected to begin later in 2002. It is a portion of that final section that is the subject of this report.

I-10 is presently elevated through the downtown area and current plans call for it to be further raised to provide adequate clearance for local streets to cross underneath. Detailed geometrics of the future mainline and ramps have been established as a part of the frontage road design. These geometrics have typically been referred to as "30% geometrics" and supplement the *General Plan*. References to the General Plan in this report are intended to encompass the changes and refinements of the 30% plans as well.

The problem addressed here is that I-10 transects Rio Nuevo, an extensive program of revitalization in the downtown area being undertaken by the City of Tucson. The freeway creates a physical and psychological barrier through the project which would be mitigated to some extent were I-10 depressed. The City of Tucson has commissioned this study to determine the viability and cost of depressing I-10 through Rio Nuevo. This report documents the methodology and findings of that study.

Current planning calls for I-10 mainline profile to be raised as much as seven feet above the existing elevated roadway, or up to 25' above natural ground in the surrounding area. The depressed profile proposed here would lower the roadway to approximately 20' below natural ground. This is a significant change in the very complex relationship between the depressed mainline, ramps and frontage roads, given the constraints of the existing frontage roads, clearance requirements, and design speed.

A major component of this study has been to develop a workable set of depressed roadway geometrics. The results of that effort are documented in the preliminary geometrics plan entitled *Casa Grande-Tucson Highway (I-10) Downtown Depression Study, January 4, 2002* that accompany this report. It is intended that these plans replace the current 30% geometrics if the proposal to depress this section of I-10 is adopted.

Extent of Depression

The proposed depression would begin south of St. Mary's Road to provide adequate clearance under a future Congress Street bridge. The depressed section would continue southward past Simpson Street. At that point it would begin to rise, returning to the elevated profile at approximately 22nd Street. The total length of deviation from the mainline profile as currently planned is about 1.3 miles. Figure 1 shows the proposed configuration of the depressed section including the bridges that would cross it, and the ramps and frontage roads that would be reconstructed.

The accompanying plans include profiles and alignments for the mainline, access ramps, frontage roads, cross-streets, and detours for mainline traffic during construction. The cross-street profiles accommodate the anticipated structural depths needed for four bridges that would cross the depressed roadway. A number of large retaining walls will be needed and a substantial amount of recently constructed frontage road will need to be rebuilt to accommodate the relatively short ramps. The following sections discuss specific issues involving the proposed mainline, ramp and frontage road geometrics.

Mainline Geometrics

Mainline Alignment. The mainline horizontal alignment is based on 30% geometrics developed with the design of the frontage roads. Note that the 30% geometrics differ slightly from the *General Plan* due to environmental constraints and clearances.

Typical Section. The typical section is also based on the 30% geometrics except that the median width has been increased to accommodate center bridge piers and wider inside shoulders recently adopted by ADOT for interstate freeways. The *General Plan* and 30% geometrics both call for a 22' wide median consisting of two ten-foot inside shoulders and a two-foot median barrier. This width has been increased to 30' to accommodate the 12' inside shoulders and 4' bridge piers with one-foot median barrier rail on each side. The widened section extends from Station 527+00 to Station 557+20, a length of 3,020'. A taper ration of 70:1 has been used to transition between the original and the widened section.

Mainline Profile. The mainline profile ties into the elevated 30% geometrics mainline profile of the at each end of the depression. Deviation from the elevated profile begins at Station 506+00.00 and extends to Station 576+50.00, a length of 7,050'. The depressed profile starts approximately 750' south of St. Mary's Road, allowing the mainline to cross over St. Mary's Road and Tucson Arroyo as currently designed. Moving southward from that point, the profile slopes downward at 2.0987% to cross under Congress Street with the lowest point along the depressed profile being just north of Congress. A new Congress Street bridge will be the northern-most crossing of the depression and would be constructed at or slightly above its current profile.

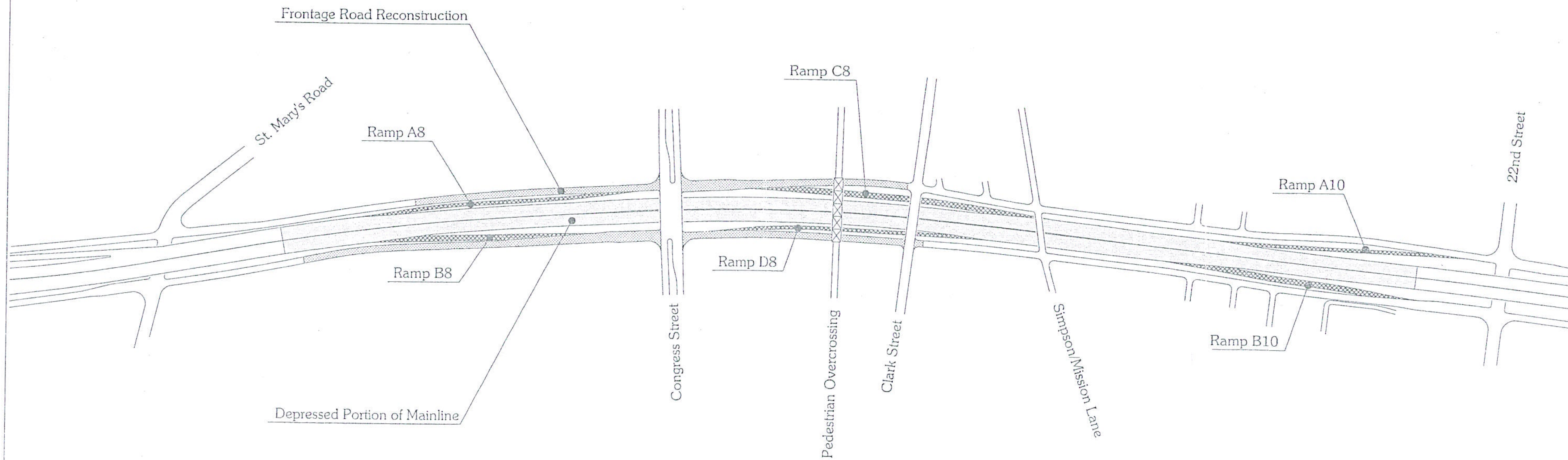


Figure 1. Limits of Proposed Depressed Section

From Congress Street, the profile slopes upward at 0.30% through the majority of the depressed section, crossing under the other three proposed bridges. Beyond the southern-most bridge at Simpson Street, the profile slopes upward at 3.4790% matching into the elevated profile approximately 530' north of 22nd Street. 22nd Street will cross under I-10 as currently planned.

RAMP GEOMETRICS

The ramps that would be impacted by the proposed depression are designated in the *General Plan* as the following:

A8	Westbound entrance ramp from Congress Street
B8	Eastbound exit ramp to Congress Street
C8	Westbound exit ramp to Congress Street
D8	Eastbound entrance ramp from Congress Street
A10	Westbound entrance ramp from 22nd Street
B10	Eastbound exit ramp to 22nd Street

The locations of these ramps are shown in Figure 1. The modifications that will be required are discussed here.

Ramp A8 (Westbound Entrance Ramp from Congress Street) It is necessary to adjust the horizontal alignment to accommodate the new vertical profile. The ramp connects to the mainline at a point where I-10 is dropping to cross under Congress Street. It is necessary that the ramp match that downward slope where it connects to the mainline before it begins rising to match into the westbound frontage road.

The length of ramp has been increased to accomplish this. The nose of the gore has been shifted northward 190' and the departure angle increased slightly to allow the ramp profile to deviate from the mainline profile more quickly and to a greater extent. The maximum horizontal shift from the original centerline is six feet which provides the needed separation.

Ramp B8 (Eastbound Exit ramp to Congress Street) The horizontal alignment of this ramp has not been modified from the original design since there is not adequate room to do so. As with Ramp A8, this ramp departs from the interstate at a downward slope. Because there is only 230' of ramp length between the mainline back-of-gore and the match point with the frontage road, the ramp cannot rise sufficiently and it is necessary to depress the frontage road profile. This and other adjustments to the frontage road profiles are addressed later.

Ramp C8 (Westbound Exit Ramp to Congress Street) The horizontal alignment of this ramp is unchanged from the current plan but the vertical alignment has been revised to address several constraints. The ramp departs from the westbound frontage road just south of the Congress Street intersection and slopes downward to the south at 2.8% to provide the necessary clearance under the pedestrian/bikeway and Clark Street bridges. Sight distance is an issue at this location due to the fact that the frontage road and ramp diverge in the opposite direction of merging traffic. To maintain safe sight-lines between the ramp and the frontage road, the ramp profile has been flattened slightly through the merge location and then dropped more steeply to reach the mainline profile.

Ramp D8 (Eastbound Entrance Ramp from Congress Street) The horizontal alignment of Ramp D8 has been modified only to accommodate the increased median width. Like C8, this ramp must also cross below the pedestrian/bikeway and Clark Street bridges but in this case the ramp profile is controlled by the mainline and the frontage road rather than the structures. The ramp and frontage road diverge in the direction of traffic so sight distance is not so critical an issue.

Ramp A10 (Westbound Entrance Ramp from 22nd Street) The length of this ramp has been increased to avoid having to lower the existing westbound frontage road at this location. The ramp departs from the mainline profile at a relatively steep uphill grade but must tie into the frontage road on a downhill grade. The resulting crest vertical curve created within the ramp body must be at least 600' long. The additional ramp length has been obtained by moving the ramp gore northward similar to the adjustment made for Ramp A8. Because of the adjustment to ramp C8, the gore-to-gore distance between C8 and C10 is actually increased from 2,138' to 2,152'.

Ramp B10 (Eastbound Exit Ramp to 22nd Street) The mainline and frontage road profile conditions described for Ramp A10 apply here as well. In this case, however, there is adequate ramp length to accommodate the resulting vertical curve without having to adjust the frontage road profile. The horizontal alignment remains as originally designed.

FRONTAGE ROAD GEOMETRICS

The *General Plan* currently calls for the mainline to be elevated as much as seven feet above the elevation of the current mainline to provide adequate clearance for roadways crossing under I-10. Weaving distance requirements between ramp terminals along the mainline limit the length of the ramps and the amount of elevation differential that can be taken up along them. The frontage roads have, in fact, been raised to accommodate the elevation constraints of the ramps. A similar situation will exist with the depressed profile except that the frontage roads need to be depressed to meet some ramp profiles. This will result in some of recently constructed frontage roads having to be re-built.

I-10 between St. Mary's Road and 22nd Street faces some of the tightest horizontal constraints of any section of interstate freeway in the Tucson area. North of Congress Street, the freeway is bounded by a historic neighborhood to the east and the Santa Cruz River Park to the west. It is these constraints that required shifting the mainline alignment and moving the frontage roads closer to the mainline. Retaining walls were constructed along the outsides of both frontage roads to prevent embankment slopes from intruding into protected or developed areas.

The limited horizontal spacing between the frontage roads and mainline forced the frontage road profiles to rise well above natural ground. To accommodate the ramps for Congress Street, for example, the frontage roads rise as much as twelve feet above ground to accommodate the ramps connecting to the future profile of I-10. This "humping" at Congress is most pronounced in the westbound frontage road profile.

With the proposed depression, a similar problem occurs only in the opposite direction. The ultimate mainline profile, instead of being approximately 20' above the cross streets, would be at least 20' below. Just as the frontage roads were raised to meet ramp profiles under the current plan, they must be lowered under the depression proposal. This section describes more specifically the frontage road changes that will be required.

Eastbound Frontage Road. The change in profile of the eastbound frontage road begins at Station 2289+00 and extends to Station 2327+50 for a total length of 3,850'. North of Congress, the profiles for Ramp B8 and the eastbound frontage road were developed concurrently to establish acceptable alignments at the merge location. The elevation of Congress Street through the frontage road intersection would not change significantly.

A portion of the existing retaining wall between the eastbound frontage road and the Santa Cruz River Park north of Congress can continue to be used with the depressed frontage road. Where the revised profile drops below the existing wall footing, the existing wall will have to be removed and a new retaining wall constructed to avoid impacting the park.

South of Congress, the eastbound frontage road profile must also drop to accommodate the departure of Ramp D8. It would match back into the existing profile at Station 2327+50 which is just south of the proposed Clark Street bridge. A new retaining wall will be required along the outside of the frontage road to avoid impacting the existing utility corridor and adjacent properties. Access between the frontage roads and some of the adjacent properties may be eliminated.

Westbound Frontage Road. The change in profile of the westbound frontage road begins at Station 2297+75 and extends approximately 3,075' to Station 2328+50, also near Clark Street. As with the eastbound frontage road, the Congress Street intersection will remain near its current elevation. South of Congress, the frontage road profile would be lowered to accommodate Ramp C8. This places the profile below the adjacent ground level and a retaining wall may be necessary along the outside of the frontage road to protect utilities and adjacent property.

Placement of Retaining Walls. While the location and height of retaining walls is tightly dictated by the roadway geometry, there is some flexibility as to where the walls are placed laterally. In some cases the choice exists whether to place the walls near the mainline or move them back toward the frontage roads. In the accompanying plans, the choice has been made to place the walls nearer the frontage roads. This tends to widen the "urban canyon" perceived by travelers passing through Tucson on I-10. It also provides the opportunity to use landscaping and other visual treatments to soften the appearance of the retaining walls. The lateral locations of these retaining walls should be evaluated in more detail during final design.

Crossing Geometrics and Structures

Bridges across the depressed section are proposed at Congress Street, Clark Street, and Simpson Street/Mission Lane. An pedestrian/bikeway overpass would also be provided between Congress and Clark. Profiles and other geometric features of the crossing structures were developed with the roadway geometrics. Structural and cost determinations were provided by Cannon & Associates. Cannon's report is attached as Appendix A.

The approximate locations of these structures are shown in Figure 1. A brief description of each is provided here.

Congress Street. A bridge at Congress Street will be a major carrier of arterial traffic. It would have three travel lanes in each direction as well as dual lanes for left turn movements entering the frontage roads. It would also have bike lanes, pedestrian areas, and a curbed median island. The total width needed for this has been assumed to be 157'-8".

The center line alignment of Congress Street does not change under this proposal. The profile is changed only slightly and should not adversely affect parcels fronting on Congress at the present time.

This structure would have two spans of approximately 96' for a total length of 196'-6". Its superstructure would be a 5'-0" deep cast-in-place post-tensioned concrete box girder deck constructed on soffit fill. Its cost has been estimated here to be $75 per square foot or a total of $2.32 million.

Pedestrian/Bikeway Bridge. The pedestrian/bikeway bridge would provide two 10' bikeways, two 8' walkways, and 8' planters each side. The bridge would span the frontage roads as well as the ramps and mainline of the freeway. The profile of the bridge would be arched such that the center is about 15' higher than the end points providing a better vantage point from the structure.

Cannon has assumed that this crossing would be a steel through-truss structure. It would have three spans of 92' in length and a fourth of 120'. The total length would be 400'-6". The total width would be 55' assuming 8' wide planters are provided. The design of the steel truss may provide sufficient visual appeal to eliminate the need for these planters.

The cost of this structure assuming planters are included has been estimated to be $120 per square foot or $2.64 million which includes the planters. Approximately $820,000 would be saved if the planters were eliminated.

Clark Street Bridge. The Clark Street bridge would be intended for local vehicular, pedestrian, and bicycle access. It would also serve as the trolley crossing and could have planters on each side. The width of structure needed for this has been assumed at 75'-8". Two spans, one at 120' and one at 140', would be needed for a total length of 264'-6". As with the Congress Street bridge, this structure would be a cast-in-place post-tensioned concrete box girder deck constructed on soffit fill. The 120' span would be 5'-6" deep and the 140' span 6-'6" in depth. The estimated cost used here is $79 per square foot including planters for a total of $1.92 million. About $400,000 could be saved by eliminating the planters, but they would be a more important visual and noise mitigation element in this case.

Simpson Street/Mission Lane Bridge. The Simpson Street/Mission Lane bridge would also serve local vehicular, pedestrian, and bicycle traffic. Its 55'-8" width would provide two 14' travel lanes, two 5' bike lanes, and two 8' sidewalks. Planters have not been assumed here. Two spans of 110'-9" are anticipated for a total span length of 226'-0". The structure would be skewed 14^{o}. It would also be a cast-in-place post-tensioned concrete box girder deck 5'-0" thick constructed on soffit fill. Its cost is also estimated at $75 per square foot or $0.94 million.

Summary. A summary of key findings from Cannon's report is provided here for convenience:

	Length	Width	Depth of Structure	Estimated Cost ($ millions)
Congress Street:	196'-6"	157'-8"	5'-0"	2.32
Pedestrian/Bikeway:	400'-6"	55'-0"	n/a	2.64
Clark Street:	264'-6"	91'-8"	6'-6"	1.92
Simpson/Mission Lane:	226'-0"	55'-8"	5'-0"	0.94

The estimates of cost are based on reasonable assumptions regarding geotechnical conditions and anticipate that the superstructures can be constructed on soffit fill. Cannon's report includes sketches of the structures which are also included in the accompanying plans.

FUTURE DECK PARK

Some discussion of providing for a future deck park between the pedestrian over-crossing and the Clark Street bridge has occurred during this study. Because the pedestrian over-crossing spans the frontage roads and Clark Street intersects with the frontage roads on grade, it would not be possible to create such a park between those structures. To develop a deck park, it would be necessary to depress the frontage roads under Clark Street as well, an approach that would increase cost and further limit access from the frontage roads. The geometric feasibility of this approach has not been explored in detail here but can be pursued during the final design if the city desires.

PAVEMENT DRAINAGE

Much of the depressed roadway is below the hydraulic grade line of the flowing Santa Cruz River. For that reason, an area of approximately 44.3 acres cannot be drained directly to the river by gravity. Three approaches for draining the depressed roadway have been considered here. A discussion of the hydrology used for this analysis is given first.

Hydrology. A 100-year design storm has been used for this analysis. Peak discharge and runoff volume were initially determined with HEC-1 using the Clark unit hydrograph as prescribed by ADOT. The equations provided by ADOT for time of concentration Tc and storage coefficient R resulted in values of 17 minutes and 0.38 hours respectively. That analysis resulted in a peak discharge of 153 cfs and a total runoff volume of 16.7 acre-feet.

A check using the City of Tucson hydrology volume resulted in the same Tc but a significantly higher peak discharge of 278 cfs and lower runoff volume of 10.6 acre-feet. The Rational Method with Tc and intensity determined in accordance with ADOT procedures produced a peak discharge of 290 cfs with Tc = 12 minutes.

For this study, the HEC-1 model has been used with R adjusted to .10 to better match the peak discharges of the city and ADOT rational methods. The result is a peak discharge of 260 cfs and total runoff volume of 17.0 acre-feet. The following table summarizes the various hydrology results. Further analysis of the hydrology should be undertaken if the depressed section is selected.

	Tc (minutes)	Peak Discharge (cfs)	Runoff Volume (acre-feet)
HEC-1 with R = 0.38 hours:	17	153	16.7
City Method	17	278	10.6
ADOT Rational Method:	12	290	--
HEC-1 with R = 0.10 hours (used here):	17	260	16.7

Direct Pumping. The first approach considered here is to pump the pavement drainage directly into the Santa Cruz River with a pumping station sized to accommodate the design peak discharge. Since function of the roadway would be dependent on the pumping system, a backup generator and redundant capacity in the form of extra pumps would be included. Maintenance would include routinely testing the pumps and exercising the generator. Additional operating cost for power would be incurred as well. Pumping can be an effective and dependable method of disposing of storm water from depressed roadways and is widely used for that purpose throughout the country.

The cost of direct pumping is estimated here to be $9.4 million. This cost is projected from work by Rust Engineering for the DLUCS II Downtown Drainage Study in 1996, increased 20% for inflation. It includes backup power and redundant capacity but no allowance for annual operating and maintenance costs.

Outfall Storm Drain. The second approach considered avoids pumping by extending an outfall storm drain sufficiently downstream that the depressed section can drain by gravity. Preliminary investigation indicates that such a storm drain sloping at 0.10% would extend from the low point of the depression just north of Congress Street downstream beyond Grant Road, a distance of 2.75 miles. The storm drain diameter would be 90" at the upstream end, but due to attenuation in the pipe could be reduced gradually to 72" at the outlet. For the purpose of estimating cost, an 84" storm drain has been assumed along the entire length.

The cost of this approach is estimated to be $5.3 million, assuming that it can be accomplished completely within existing rights-of-way. This approach offers both a lower initial cost than the pumping alternative and it eliminates the associated annual operating and maintenance cost. It does, however, introduce an number of other complexities such as determining a suitable route within existing right-of-way, resolving likely conflicts with cross-drainages entering the Santa Cruz River from the east, and crossing under arterial roadways. This is the most problematic of the three approaches considered here and should be investigated more thoroughly before this approach is adopted.

Storage. The third alternative considered here is to store the design storm below the freeway in a concrete vault. A 6 cell 12' x 12'x 840 ADOT standard RCBC would, for example, provide adequate volume to do this. A relatively small pump station would be used to drain the vault subsequent to the storm event. A pumping station of 10 cfs capacity would handle this volume in less than 24 hours. Redundancy and backup power would likely be used but would not be as critical as in the case of the direct pumping alternative. The cost of this approach is estimated to be $3 million.

Recommendation. Because it is the least expensive and the simplest, the storage approach is recommended here. The use of multiple vaults and pump stations, or the use of a single barrel vault that would serve as the storm drain main may prove even more cost-effective and should be explored during final design if the depressed section is pursued.

OFFSITE DRAINAGE

Five offsite drainages cross I-10 within the limits of the depression. The concept proposed here is that existing culverts will be replaced with inverted siphons. The capacities of these crossings would be increased to prevent breakout flows from entering the depressed section in a 100-year storm. Of particular concern is breakout flow accumulating to approximately 400 cfs currently crossing the freeway through the Congress Street overpass.

Though not directly affected by the depressed profile, special measures such as low floodwalls may be required at Tucson Arroyo as well.

For the purpose of estimating cost, the size of the new siphons has been assumed to be twice the existing culverts. This would presumably eliminate breakout and provide for additional losses through the siphons.

The residual water in the siphons would be drained by one or more small pumping stations. The pumps used for this purpose could be housed in the same structure but would be isolated from the pavement drainage system to prevent offsite flow from backing up onto the freeway.

The cost estimate anticipates concrete lining for Simpson Wash and 18th Street Wash from I-10 to the Santa Cruz River and $240,000 for additional drainage easements for the downstream channels. A new box culvert for Mission Lane has also been included.

UTILITIES

Relocation of utilities crossing I-10 at Alameda Street, Congress Street, Clark Street, Simpson Street/Mission Lane, 18th Street, and 19th Street will be necessary. Existing utilities in the frontage roads will also need to be relocated for the drainage structures and possibly where the frontage roads are to be lowered. Realignment of sanitary sewers is likely to be required. The cost estimate includes $1 million for utility relocation.

GROUNDWATER INFILTRATION

The potential for infiltration of groundwater from the Santa Cruz River impacting the depressed roadway has been investigated by URS Corporation. This study has been performed to determine if increased pavement thickness or tie-downs will be necessary to resist the upward force of hydrostatic pressure. Retaining walls could be subjected to increased lateral forces as well.

The current practice for retaining walls is to provide drains to prevent the buildup of hydrostatic pressure rather than resist the force structurally. This is now routinely accomplished by placing geocomposite material along the back face. Seepage reaching the wall flows downward to the base of the wall where it can be drained into the roadway through weep holes or collected in perforated pipes that discharge into the pavement drainage system.

A similar approach is also used for the pavement structure. An underdrain system consisting of an open-graded aggregate base course or of geocomposite material allows upward flowing water to drain out from under the roadway. With aggregate base, a cloth filter must be used to prevent upward migration of fines from contaminating the base. The use of geocomposite material is more expensive but adds to the strength of the base resulting in an offsetting reduction in the cost of the pavement structure. ADOT allows the R-value to be increased by 10 when geocomposite material is used.

URS performed a flow net analysis to estimate the maximum rate at which groundwater that can be expected to reach the depressed roadway. Using a variety of assumptions, infiltration rates ranging from 95 gpm to 2,286 gpm were found. The upper limit assumes that a state of steady flow is achieved at a flow depth of 20' in the river and that the material between the river and the depressed section is uniformly graded sand, both considered to be conservative assumptions. In any event, 2,286 gpm is less than 6.5 cfs, an amount the pavement drainage system can accommodate without an increase in capacity or cost. This analysis is described in more detail in URS's report attached here as Appendix B.

Potential for Contamination

URS also investigated the potential for groundwater infiltration becoming contaminated as it moves from the river to the depression. Contaminated flow could require special treatment to be discharged back into the river, adding both cost and complexity to the project. A review of data from monitoring wells in the area shows violation of current drinking water standards in the area do not exist. This generally suggests that migration of contaminants would not occur, particularly for the transient event of a Santa Cruz River flow. A former gas station and a leaking underground storage tank are known to exist between the river and the depressed section, however, and more testing and analysis should be performed if the proposal for the depressed section is adopted. URS's preliminary analysis is also described in their report in Appendix B.

Retaining Walls

Heights and locations of retaining walls needed to construct the depressed section have been determined in conjunction with developing the roadway geometrics. Elevations of the necessary retaining walls are included in the roadway geometrics attached plan set. In all, approximately 12,200 linear feet of retaining wall ranging from 13' to 31' in height are envisioned. The total area of retaining wall is currently estimated to be 223,600 square feet.

URS has evaluated various retaining wall systems given the right-of-way constraints and recommended that soil nailing be utilized. The basic soil nailing wall system is estimated to cost $40 per square foot which would include the drainage measures. Pre-cast or cast-in-place facing can be expected to add $10 to $15 per square foot, depending on the visual treatment that is employed. A total cost of $50 per square foot has been assumed for this study. More explanation of the soil nailing system including diagrams is also found in URS' report in Appendix B.

CONSTRUCTION SEQUENCING

To maintain access and movement of traffic along the corridor, the depressed section of I-10 will need to be built in phases. A sequencing plan consisting of three phases is presented in the attached plans. These phases are discussed here.

Phase 1. Frontage Roads. The first phase consists of reconstructing the frontage roads to the lower profiles where necessary. Retaining walls needed between the frontage roads and the ramps or interstate would also be built at this time. The specific steps of Phase I are as follows:

1. Close the frontage roads to through traffic in order to remove the existing pavements and retaining walls where necessary. It may be necessary to maintain local traffic in certain sections.
2. Construct new retaining walls adjacent to the frontage roads and ramps. Depending upon the construction techniques utilized and the extent of walls built, temporary shoring may be required between the walls and I-10. As much of the retaining wall as possible should be built at this time to limit the extent of impact to traffic during the next phases of work.
3. Construct the new frontage roads including the portions of the new ramps at the frontage road terminals. The extent of pavement placed at this time will depend upon the walls built to that point and on the impact to the I-10 traffic.
4. Construct the temporary detours between the frontage roads and existing I-10. If the ultimate profile of I-10 has been constructed prior to this project, longer detours than those shown in the plans will be required to tie into the wider roadway section and higher profile of the reconstructed interstate.
5. Install temporary signing and pavement marking for detouring mainline traffic onto the frontage roads.

Phase II. The second phase will be the major phase of construction and will take the longest period of time. While this work is in progress, mainline traffic will be routed along the frontage roads in similar fashion to the current plan. Since FHWA does not allow interstate traffic to be stopped with traffic signals, Congress Street will be closed to through movements across the interstate. The other steps in this phase are:

1. Close Congress Street and Clark Street to through movements. Right turns from the frontage roads onto the side streets and from the side streets onto the frontage roads would be allowed unless an unusually high number of accidents or congestion associated with these movements is experienced.
2. Reroute mainline traffic to the frontage roads.
3. Remove existing bridges and retaining walls along the corridor as required. This may require temporary shoring along the existing frontage roads in order to eliminate the undermining of the frontage road pavements.

4. Remove existing mainline pavement and fill. Excavate to the proposed grade line for the depressed freeway except where cast-in-place concrete bridges are to be located. A large amount of material will need to be moved out of the work zone across the frontage roads as they carry detoured mainline traffic. The feasibility of an alternate route for removing this material, such as temporary tunnels under the frontage roads, should be investigated during final design.

5. Construct the new cross drainage structures, the pavement drainage system, and underground utilities crossing the depressed roadway.

6. Construct the remaining retaining walls and new bridges within the depressed section. Excavate under the bridge decks once they are complete. Construct the portions of the pedestrian overpass spanning the mainline and ramps at this time. Construct the portions spanning the frontage roads at this time or later in Phase III if preferable.

7. Place mainline pavement. Place ramp pavement to the extent that the retaining wall construction to date allows.

8. Place pavement markings and signage along the mainline. Leave temporary barriers across ramps that cannot be opened due to remaining work to be performed on those ramps.

Phase III. The final phase of work consists of constructing the balance of retaining walls and ramps not built during the previous phases. It also includes final cleanup of the project area. The specific steps are:

1. Shift through-traffic from the frontage roads to the reconstructed mainline.

2. Close the frontage roads to through traffic.

3. Remove the detour pavements and fill material. Restore these areas to the original ground line and replace any landscaping necessary. Remove the temporary frontage road pavement marking and replace with the original striping.

4. Construct the remaining sections of retaining wall and ramp pavement.

5. Install the final signing and pavement markings and open all frontage roads and ramps to traffic.

COST ESTIMATE

Costs have been estimated for the following scenarios:

1. The elevated section is built as currently planned.
2. The depressed section is built in lieu of the elevated section.
3. The elevated section is built as currently planned with the depressed section constructed at a later date.

A summary of these estimates shown in $millions is as follows:

	Cost to ADOT	Cost to City	Total Cost
Elevated Roadway as currently planned:	25.9	--	25.9
Depressed roadway constructed now:	25.9	50.6	76.5
Depressed roadway constructed in future:	25.9	78.3	104.2

The cost estimates are detailed in Appendix C. These estimates include allowances of 15% for contingency, 15% for design, and 15% for construction administration.

Appendix A
Report by Cannon & Associates

Cannon & Associates, Inc.

Consulting Engineers
406 South 4th Avenue
Tucson, Arizona 85701

(520) 792-2200
(520) 792-3668 FAX

November 30, 2001

Mike Johnson, P.E.
Johnson-Brittain & Associates, Inc.
378 North Main Street
Tucson, Arizona 85701

RECEIVED
DEC 03 2001
Johnson-Brittain & Associates

Re: I-10 Conceptual Design
CA 00046-01

Dear Mike:

Cannon & Associates, Inc. has completed the structural concept design of four underpasses that will carry local traffic over Interstate 10. These structures would be needed if I-10 were depressed within the vicinity of the Rio Nuevo project. The four proposed underpasses will be at Congress Street, Simpson Street, Clark Street and a pedestrian crossing that will be located between Congress and Clark Street. We have performed preliminary calculations to determine the sizes and types of the critical structural elements so that a preliminary cost estimate could be developed for each bridge. We understand that the intent of our work is to enable you to develop a preliminary cost estimate for the City of Tucson so that they can evaluate the feasibility of constructing this project.

The assumptions made to determine the structural conceptual design of the underpasses and to develop the cost estimates were as follows:

1. Interstate 10 will be closed during construction of the underpasses. All traffic will be detoured onto the I-10 frontage roads. Therefore, it is possible that the underpasses can be constructed on soffit fill. Bridges constructed on soffit fill are cheaper than bridges constructed on falsework, and in many cases are cheaper than precast bridges.
2. It is possible that groundwater will be encountered during construction since the depressed I-10 profile is near the thalweg of the Santa Cruz River. We have therefore assumed that some dewatering will be required during construction of the abutment and pier foundations. We estimate that dewatering will add approximately \$1 to \$2 per square foot to the cost of each underpass.
3. Shoring will be required to support the excavations for the full-height abutments. Shoring will cost approximately \$20 per square foot of shoring.
4. Geotechnical recommendations regarding foundation design were not provided at this concept level. We are assuming based on past experience that the bearing capacity of the soil will be adequate so that spread footings could be used to support the underpasses.

We are enclosing Sheets 1 of 2 and 2 of 2 showing the proposed underpasses. A description and cost estimate of each follows:

Congress Street Underpass (I-10 Station 530+26.95):
The Congress Street Underpass will be a two-span cast-in-place prestressed post-tensioned concrete box girder underpass. The span lengths will be approximately 96' each and the total structure length will be 196'-6". The 5'-0" deep box girder superstructure will be constructed on soffit fill. The abutments will be full-height concrete walls supported on spread footings. The center pier will be located within the I-10 median and will consist of at least 3 concrete columns supported on spread footings. The structure will be 157'-8" wide and will accommodate 6 travel lanes, 4 left turn lanes, two bike lanes, a median and two sidewalks. See Sheet 1 of 2. The estimated cost of this structure is as follows:

Congress Street Underpass

Total Area of Superstructure:	30,982 Sq. Ft.
Estimated cost per square foot:	$75
Total estimated cost:	$2,323,650

Pedestrian Crossing: (I-10 Sta 540+55.48):
The pedestrian crossing will be a four-span bridge that crosses I-10, the I-10 entrance and exit ramps and the frontage roads. Three of the spans will be 92' and the fourth span will be 120' long. The total structure length will be 400'-6". The bridge will be approximately 55' wide which includes two 8' wide planters, two 8' sidewalks, and two 10' bike lanes. If the planters are deleted, the bridge width will be approximately 38'. The abutments will be full-height concrete walls supported on spread footings. Piers 1 and 3 will be located between I-10 and the entrance/exit ramps. Pier 2 will be located within the I-10 median. Each pier will most likely consist of a single pier column supported on a spread footing.

Since this underpass will cross over the frontage roads, the superstructure will either need to be constructed on falsework or be premanufactured and erected at one time with a crane. Precast concrete girders or steel trusses could be used for this crossing. Cannon & Associates, Inc. recommends that the underpass be constructed using steel trusses. The trusses could be prefabricated and shipped to the site. They would then be field-bolted together and each span would be lifted into place at a time. This operation could take place at night and the frontage roads would only need to be closed during erection of the trusses. The structure would have a concrete deck supported by steel stringers and floor beams that span between the trusses. The flexibility in the shape of the steel trusses gives the City of Tucson an opportunity to provide an aesthetically pleasing structure in this high profile area. See Sheet 1 of 2. The estimated cost of this structure is as follows:

Pedestrian Crossing	With Planter	Without Planter
Total Area of Superstructure:	22,028 Sq. Ft.	15,219 Sq. Ft.
Estimated cost per square foot:	$120/Sq. Ft.	$120/Sq. Ft.
Total estimated cost:	$2,643,360	$1,826,280

Clark Street Underpass (I-10 Station 543+87.01 or 545+24.37):
The Clark Street Underpass will be a two-span cast-in-place prestressed post-tensioned concrete box girder underpass. The span lengths will be approximately 120' and 140' and the total structure length will be 264'-6". The box girder superstructure will be approximately 6'-6" deep if planters are used and approximately 5'-6" deep without planters. The bridge will be constructed on soffit fill. The abutments will be full-height concrete walls supported on spread footings. The center pier will be located within the I-10 median and will consist of 3 concrete columns supported on spread footings. The structure will be 91'-8" wide which includes two 8' wide planters, two 8' sidewalks, two 5' bike lanes, two 14' travel lanes and one 20' median with trolley tracks. If the planters are deleted, the bridge width will be 75'-8". See Sheet 2 of 2. The estimated cost of this structure is as follows:

Clark Street Underpass	With Planter	Without Planter
Total Area of Superstructure:	24,247 Sq. Ft.	20,015 Sq. Ft.
Estimated cost per square foot:	$79/Sq. Ft.	$75/Sq. Ft.
Total estimated cost:	$1,915,513	$1,501,125

Simpson Street Underpass (I-10 Station 553+14.39):
The Simpson Street Underpass will be a two-span cast-in-place prestressed post-tensioned concrete box girder underpass. The span lengths will be approximately 110'-9" each and the total structure length will be 226'-0". The 5'-0" deep box girder superstructure will be constructed on soffit fill. The underpass will cross I-10 on about a 14° skew. The abutments will be full-height concrete walls supported on spread footings. The center pier will be located within the I-10 median and will consist of 2 concrete columns supported on spread footings. The structure width of 55'-8" will accommodate two 8' sidewalks, two 5' bike lanes and two 14' travel lanes. See Sheet 2 of 2. The estimated cost of this structure is as follows:

Simpson Street Underpass

Total Area of Superstructure:	12,581 Sq. Ft.
Estimated cost per square foot:	\$75/Sq. Ft.
Total estimated cost:	\$943,575

We have also estimated that it would cost approximately \$3,200 per lineal foot to construct a continuous full-height abutment that could be used for a future bridge between the Clark Street and Pedestrian Crossing Underpasses. This estimate was based on an abutment supported on spread footings with a height of approximately 26' from the top of the abutment to the finished roadway of Interstate 10.

The cost estimates provided in this letter do not include mobilization, design engineering, or construction engineering.

If you have questions or need additional information, please call.

Sincerely,

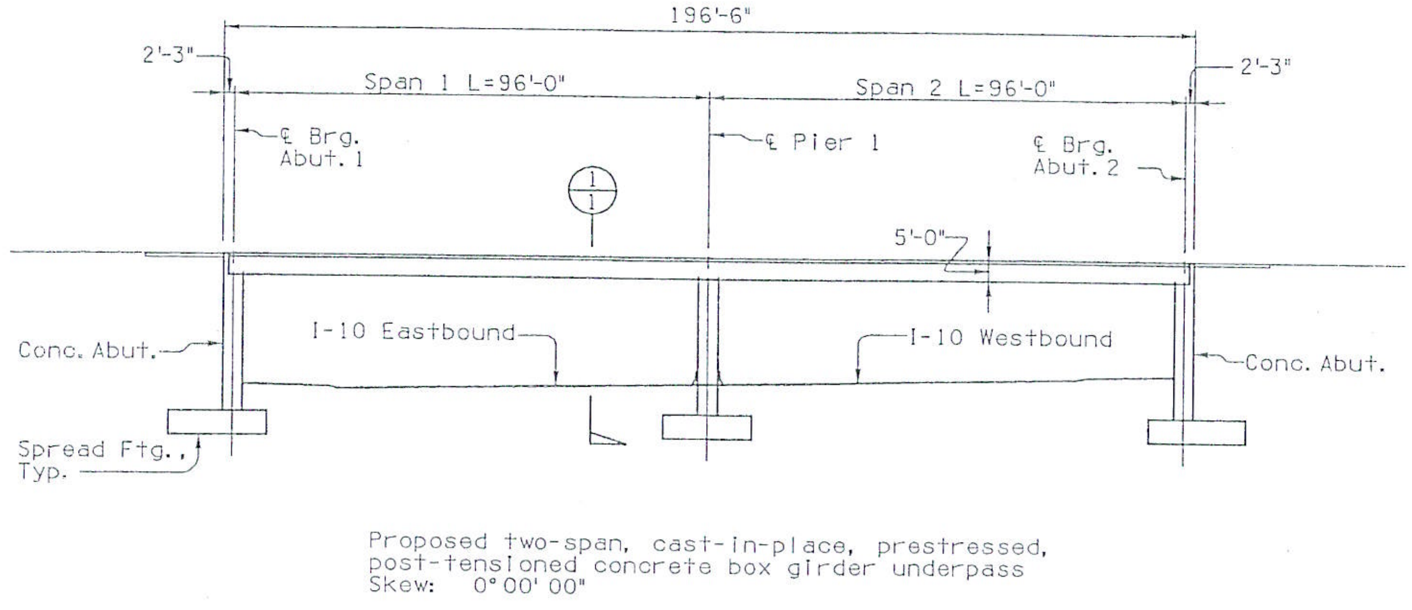
Ted W. Buell

Ted W. Buell, P.E.
Project Engineer

Reviewed by,

Jerry A. Cannon, P.E.
Project Manager

Registered Professional Engineer (Structural)
CERTIFICATE NO.
6161
JERRY A.
CANNON
Date signed 11/27/01
ARIZONA, U.S.A.



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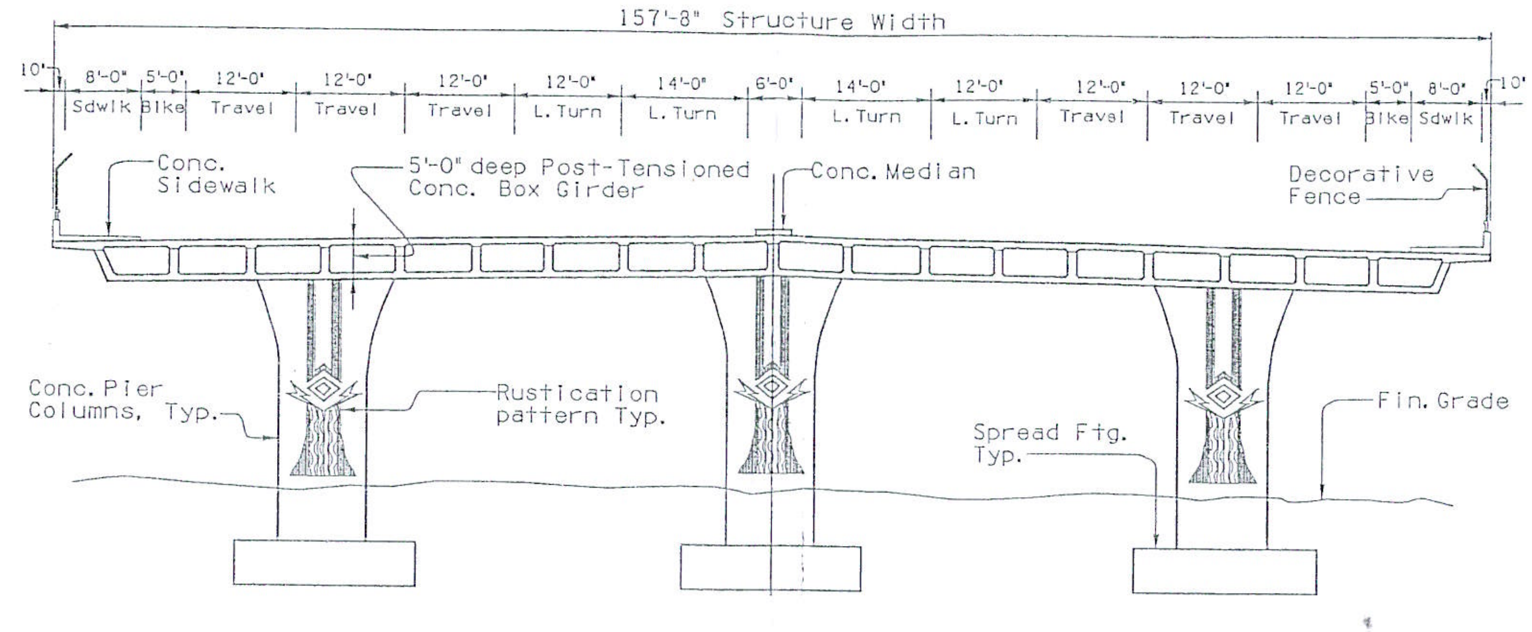

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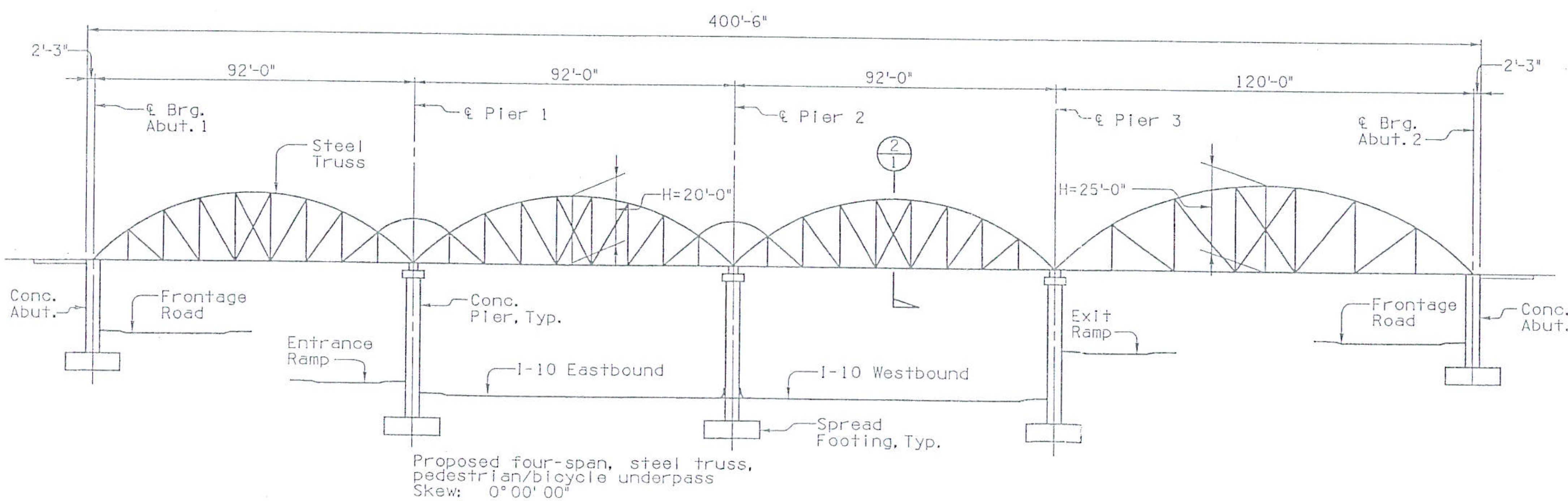

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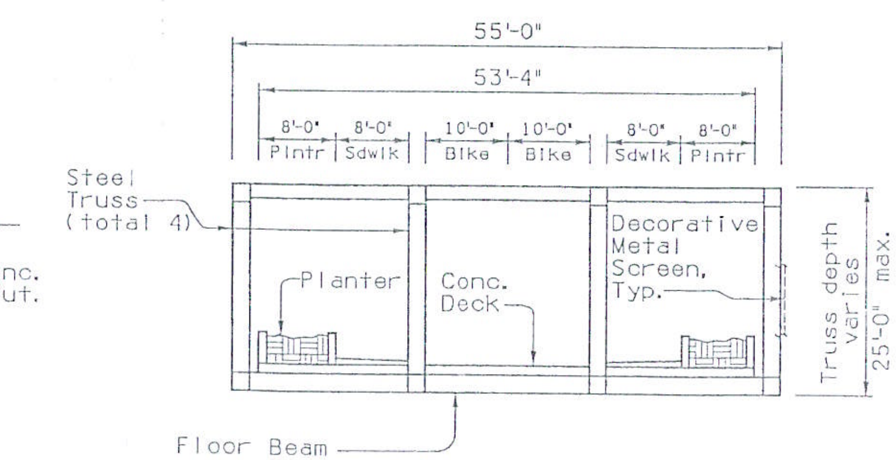

TYPICAL SECTION @ PED. CROSSING
3/32" = 2'-0"
2

SHEET 1 OF 2

PRELIMINARY
NOT FOR CONSTRUCTION
CONCEPTUAL DESIGN

	NAME	DATE
DESIGN	TB	11/01
DRAWN	VG	11/01
CHECKED	TB/JAC	11/01

Cannon & Associates, Inc.
Consulting Engineers CA 00046-1

I-10 CONCEPTUAL DESIGN

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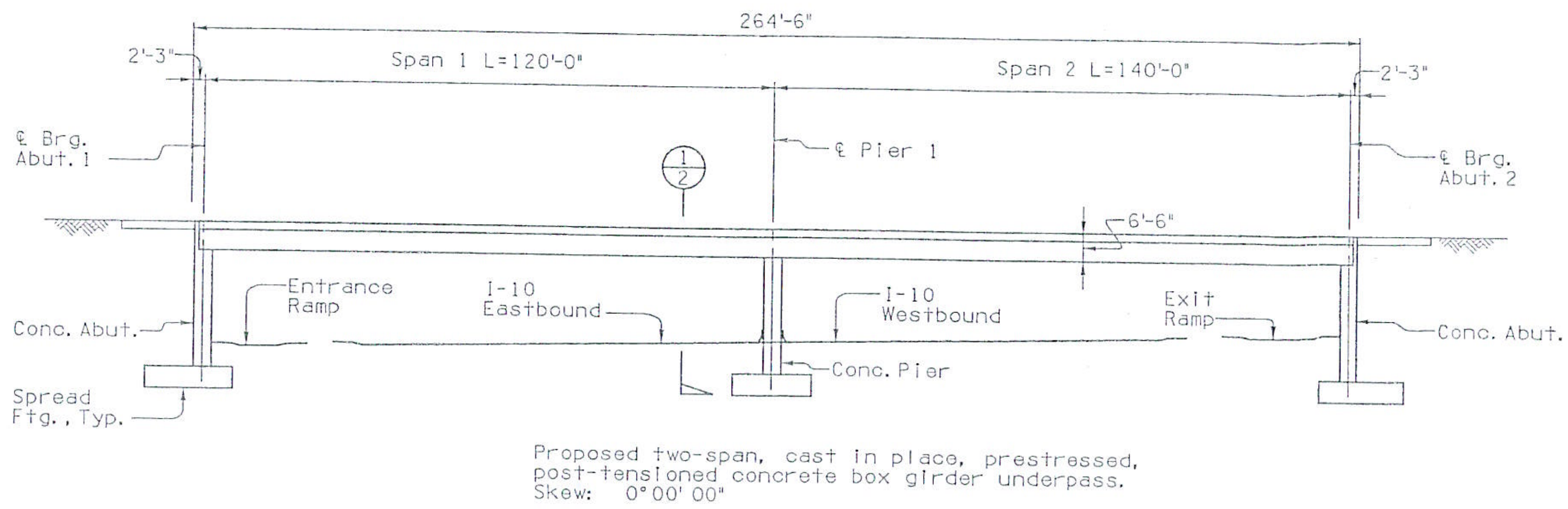

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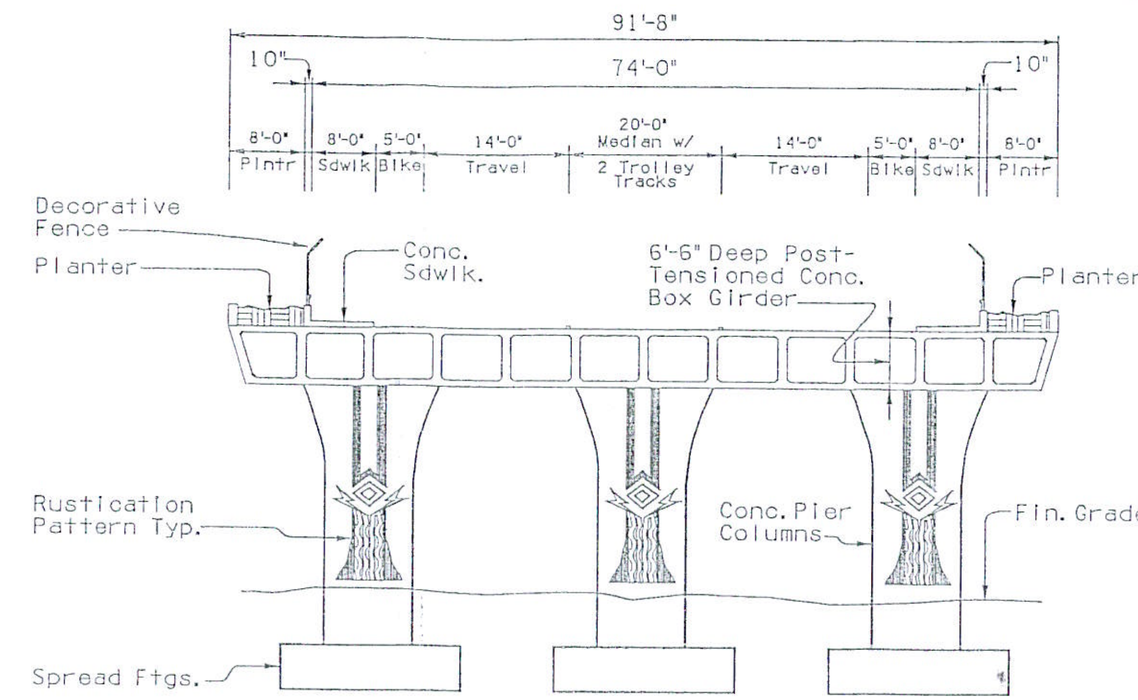

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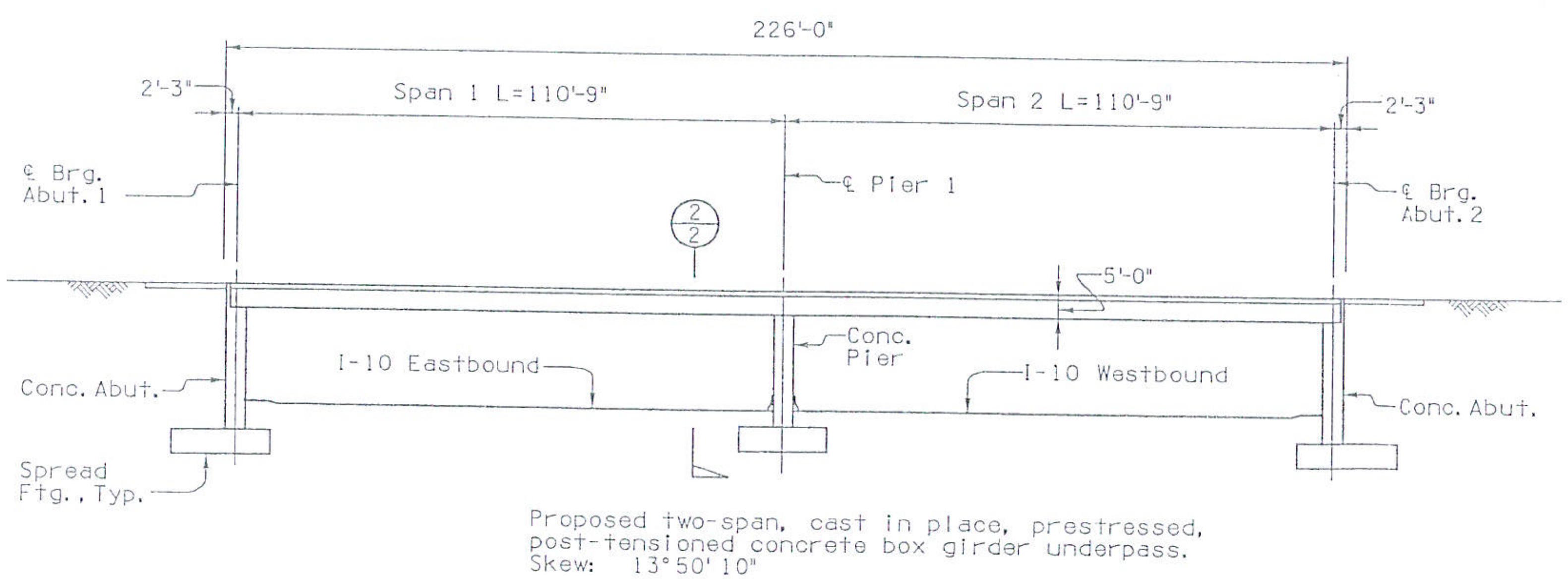

SECTION @ CONSTR. ℄-SIMPSON ST. (I-10 Sta. 553+14.39)
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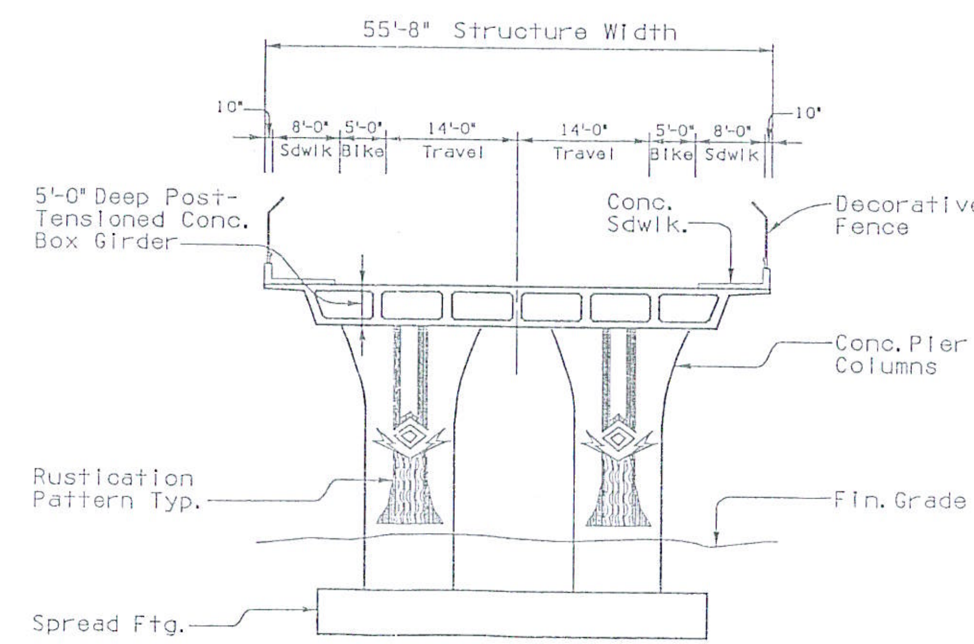

TYPICAL SECTION @ PIER-SIMPSON ST. (2)
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PRELIMINARY
NOT FOR CONSTRUCTION
CONCEPTUAL DESIGN

	NAME	DATE
DESIGN	TB	11/01
DRAWN	VG	11/01
CHECKED	TB/JAC	11/01

Cannon & Associates, Inc.
Consulting Engineers CA 00046-I

SHEET 2 OF 2

I-10 CONCEPTUAL DESIGN

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Appendix B

Report by URS Corporation

URS

GEOTECHNICAL STUDY
I-10 DEPRESSED SECTION

Prepared for
JOHNSON BRITTAIN
AND ASSOCIATES, INC

URS Job No. E2-00002009.00
December 31, 2001

4742 North Oracle Road, Suite 310
Tucson, AZ 85705

TABLE OF CONTENTS

Registered Professional Engineer (Civil) CERTIFICATE NO. 32901 N. C. SAMTANI Date Signed 12-31-01 ARIZONA, U.S.A.

Registered Professional Engineer (Civil) CERTIFICATE NO. 11595 EDWARD A. NOWATZKI Date Signed 12-31-01 ARIZONA, U.S.A.

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

To accommodate future developments in and around downtown Tucson, the City of Tucson (COT) and Arizona Department of Transportation (ADOT) are considering depressing the Interstate 10 (I-10) corridor between approximately St. Mary's Road and 22nd Street. Figure 1.1 shows the proposed change in grade as compared to existing I-10 corridor. Walls will be used to retain the cuts for the depressed roadway.

An aerial photograph of the site is shown in Figure 1.2. As shown in Figure 1.2, the depressed roadway is in close proximity to the Santa Cruz River. The distance from the roadway to the east bank of the river channel varies from approximately 300 to 800 feet along the depressed section. The deepest portion of the depressed roadway will be between Congress Street and Simpson Street where the roadway elevation will be approximately three (3) feet below the current elevation of the Santa Cruz River bed.

As part of the evaluation of the feasibility of the depressed section, URS Corporation (URS) was contracted by Johnson Brittain and Associates, Inc. (JBAI) to perform the following tasks:

1. Conduct a preliminary study to investigate the critical case for upper limits of input parameters to arrive at an order of magnitude value for potential seepage flow. The preliminary estimates of potential seepage will be used to evaluate the feasibility of constructing the depressed roadway in close proximity to the Santa Cruz River bed. If the preliminary study indicated that seepage flows were potentially manageable, final roadway design would require additional modeling with more detailed input parameters to refine estimates of seepage flow.

2. Review available literature to determine possible contamination in subsurface flows from potential historic sources in the proximity of the project, e.g., old landfills, service stations, etc.

3. Evaluate feasible retaining walls and drainage systems and measures to mitigate possible buoyancy of the depressed roadway including preparation of cost estimates.

A summary of our approach used to perform the above tasks and the layout of this report is described below.

1.1 Approach

As a first step in this task, URS compiled and reviewed existing data and reports that describe the lithology and hydraulic properties of sediments in the project area. The results of this review were used to develop a conceptual model for three representative cross sections for the analysis of seepage from the Santa Cruz River into the proposed depressed section. A commercially available computer code, SEEP/W (GEO-SLOPE International Ltd., 1998), was used to obtain estimates of potential flow quantities that could reach the retaining wall drains and roadway underdrains and to obtain estimates for hydrostatic pressure beneath the roadway with and without a geocomposite underdrain system. Simulations were conducted for a reasonable range of values for the input variables of hydraulic conductivity, vertical anisotropy, and river stage. Section 2.0 of this report presents the results of these analyses.

URS researched and reviewed existing data on possible contamination from historic sources in the proximity of the project, e.g., old landfills, services stations, etc. Results of our literature search are discussed in Section 3.0 of the report.

URS evaluated the most appropriate retaining wall systems based on our understanding of the subsurface lithology as derived from our literature review and on geometric constraints imposed by adjacent structures including the I-10 frontage roads that might impact construction techniques. Two retaining wall systems appear to be most appropriate: a soil nail wall system and a soldier pile lagging system. The features of both of these systems are described in detail below and cost estimates are provided based on recently completed projects. Section 4.0 of the report presents a discussion of the retaining walls.

Subsurface drainage systems for the retaining walls and underdrain systems for the roadway pavement were also evaluated. Geocomposite drains were found to be the most cost effective for either of the retaining wall systems recommended. A preliminary design of the roadway underdrain system consisting of an open-graded aggregate base (OGAB), with or without a geocomposite drain, was performed for seepage flow quantities determined from SEEP/W runs. Details of the roadway drainage systems are provided in Section 5.0 of the report.

2.0 ESTIMATION OF SUBSURFACE FLOWS

2.1 Model Description - General

Potential seepage from the Santa Cruz River to the proposed I-10 depressed section was modeled using SEEP/W (GEO-SLOPE International Ltd., 1998). SEEP/W is a finite element software package that can be used to model saturated and unsaturated water movement and pore pressure distribution within porous materials.

2.1.1 Input

Geometry

The geometry of the problem must be defined as the first step in the use of SEEP/W. Because of a bend in the Santa Cruz River in the project area, the depressed roadway was divided into three sections that represent horizontal distances of 300, 500, and 800 feet from the roadway to the river channel (Figure 1.2). The different lengths for the 300-ft and 800-ft reaches correspond to different river stages. A geometric model representing a vertical cross section, one foot in horizontal thickness, was constructed for each of the three sections. The three cross-sections are shown on Figure 2.1. These three cross-sections were used throughout the study. Only the hydraulic parameters and the boundary conditions were varied.

Boundary Conditions

Various boundary conditions can be set to model known or anticipated values of flow and/or pressure. The most commonly known boundary conditions are "zero flow" boundaries, e.g., soil cement bank protection; "zero pressure head" boundaries, e.g. the free water surface in the river channel; the groundwater surface; and a "zero elevation reference" that can be chosen arbitrarily. An example of boundary conditions is shown on Figure 2.2.

Hydraulic Parameters

The following hydraulic parameters are required as input to the SEEP/W program:

- hydraulic conductivity,
- vertical anisotropy, and
- constant head pressure representing river stage.

Hydraulic Conductivity

Hydraulic conductivity can be expressed as isotropic saturated conductivity (K); anisotropic conductivity consisting of vertical conductivity (K_v) and horizontal conductivity (K_h); or unsaturated conductivity (K_{unsat}). Vertical anisotropy is defined as the ratio K_h/K_v. The model requires definition of terms in a functional relationship to express the type of conductivity being considered. Because the model simulates saturated and unsaturated flow, model input requires a hydraulic conductivity function that allows unsaturated hydraulic conductivity (K) to change with moisture content. Two cases for hydraulic conductivity were simulated. The upper critical case was simulated by using values provided by the model SEEP/W for uniform or poorly graded sand. A well graded sand has smaller particles partially blocking voids between larger particles resulting in lower effective porosity, smaller diameter pore pathways, and lower hydraulic conductivity. Sand particles of uniform size stack together to create larger effective porosity, larger diameter pore pathways, and larger hydraulic conductivity. Of the hydraulic conductivity functions provided with the SEEP/W model, the "Uniform Sand" function represented the largest value.

Because heterogeneity of the sediments likely results in lower hydraulic conductivity, a second material was simulated with values for saturated hydraulic conductivity that were approximately an order of magnitude smaller than values for uniform sand. In most places in the Tucson Basin, stream channel alluvium is unsaturated. Hydraulic parameters for saturated conditions cannot be obtained from test wells and data from core samples are not widely available. Values for the second material were based on hydraulic conductivity measured in core samples along the Rillito River by the U.S. Geological Survey (Hoffman and Ripich, 2001). The unsaturated hydraulic conductivity function for the second material was based on measurements of moisture content and matric potential in the core samples, and on similar unsaturated hydraulic conductivity functions given by SEEP/W. Values for the second material were selected because they were approximately an order of magnitude smaller than those for uniform sand used for the upper critical case and because they were based on measured values for sediments from a similar depositional environment.

Vertical Anisotropy

Vertical anisotropy for hydraulic conductivity sometimes occurs in alluvial depositional environments where sediments with different hydraulic properties are deposited in horizontal layers. Fine-grained sediments interlayered with coarse-grained deposits may result in larger horizontal hydraulic conductivity (K_h) than vertical hydraulic conductivity

(K_v). To investigate model sensitivity to the effect of vertical anisotropy, the ratio K_h/K_v was simulated for values of 1 and 10.

River Stage

River stage at peak flow for the period of record is shown on Figure 2.3. Model sensitivity to river stage was investigated by simulating river stage depths of 5 and 20 feet to approximately bracket the range for river stage during peak flow events (Figure 2.4). River stage at 20 feet is roughly equivalent to bank-full flow conditions. Bank protection along this reach of the Santa Cruz River was designed to contain a 10-year flood event (Yash Desai, City of Tucson Floodplain Division Manager, personal communication).

2.1.2 Output

Flow Quantity

Model output for flow is in units consistent with the units input for geometry and hydraulic parameters. In this study, the input units for geometry and hydraulic parameters were chosen so that output flows would be in cubic feet per second per foot. The output for each of the three sections considered in this study, when then multiplied by the corresponding length of that section, results in the flow from that section in cubic feet per second. The total flow into the depressed section is simply the sum of the flows from the three modeled sections.

Total Head

Total head equals pressure head plus elevation head. Model output for total head is in units consistent with the units input for geometry and hydraulic parameters. In this study, the input units for geometry and hydraulic parameters were chosen so that output for total head would be in units of feet. The total head at any location can be converted to pressure head by subtracting from it the known elevation head at that location. Pressure head in feet can be converted to units of pounds per square foot simply by multiplying the value of pressure head by 62.4, the unit weight of water in pounds per cubic foot.

Graphical Representations

SEEP/W can be used to generate flow nets that show flow lines (velocity vectors for each element of the net) and equipotential lines (lines of constant total head) from one flow boundary to the other. SEEP/W can also be used to generate plots that show lines of equal pressure throughout the flow section. These can be used to estimate pressures along "zero flow" boundaries, as for example, along the bottom of the roadway section.

2.1.3 Assumptions

The model is considered to be conservative because of the following simplifying assumptions:

1 Sloping roadways were not accounted for. The road base is assumed to be 10 feet below the riverbed at all locations below river stage. When the river stage is increased from 5 feet to 20 feet, the length of the depressed section where seepage occurs increases from 3,650 feet to 4,900 feet. The expanded area resulting from a river stage of 20 feet is shown with dashed lines in Figure 1.2.

2 The model simulates steady state conditions. In most cases, flow in the river is of short duration and steady state conditions may not be achieved.

3 Vertical drainage into underlying basin-fill sediments was not modeled. In reality, a significant portion of groundwater recharge from the Santa Cruz River will enter underlying basin-fill sediments. In the conceptual model for this project, these sediments are modeled as a no flow boundary.

4 Tributary washes that impact the study area were considered lined so that their effect is included in the river stages considered, i.e., there is no seepage taking place from the washes themselves.

2.1.4 Potential Model Refinements

Future refinements to the simulations could include:

1 Construction of additional models to further discretize the area to more accurately account for variations in distance to the river and changing roadway elevations.

2 Modeled horizontal layers with variations in hydraulic properties that correspond to lithologic logs for a larger vertical section including the upper part of the regional aquifer.

3 Transient model simulations that incorporate an analysis of frequency, duration and magnitude of flow in the Santa Cruz River and its tributaries.

4 Refinement of the model to simulate conditions on the retaining wall of the depressed section to represent a vertical seepage face at atmospheric pressure.

2.2 Case I – Flow Into Open Roadway

2.2.1 Geometry

A finite element mesh consisting of 10ft x 10ft elements was constructed using SEEP/W for each of the three river segments (Figure 2.1). Hydraulic conductivity, vertical anisotropy, and pressure from river stage were varied to simulate anticipated conditions and to perform sensitivity analyses.

2.2.2 Boundary Conditions

For conceptual modeling of seepage to the proposed I-10 depressed section, all model boundaries were treated as no flow boundaries except for nodes representing the floor of the Santa Cruz River channel, and nodes representing the depressed section of the roadway (Figure 2.2). Existing bank protection along the Santa Cruz River was modeled by a vertical line of no flow nodes extending 5 ft below the riverbed. The pressure head at all nodes in the depressed section equals zero. Therefore the total head for all nodes on the floor of the depressed section was set at a value of negative 10 ft, representing the 10 ft vertical distance from the bed of the river channel (arbitrary zero elevation reference datum) to the floor of the depressed roadway section. The retaining wall of the depressed section represents a vertical seepage face where the distribution of total head is difficult to model. For the purposes of this study the total head at each vertical node on the wall of the depressed section was set equal to zero.

2.2.3 Hydraulic Parameters

Model flow rates were compared using a horizontal hydraulic conductivity of 3.3E-4 or 5.15E-5 ft/sec, K_{unsat} function derived from Rillito River data and SEEP/W uniform sand data, and vertical anisotropy ratio $K_h/K_v = 1$ and 10. River stage was simulated at 5 feet and 20 feet.

2.2.4 Results

Model output and computed flows for all simulations are presented in Tables 2.1 and 2.2. Results for Case I are summarized in Table 2.3. Model sensitivity to input parameters was demonstrated by varying each parameter one at a time for constant geometry and boundary conditions. Model sensitivity is shown graphically on Figure 2.4. Results indicate that the model output for flow is slightly more sensitive to hydraulic conductivity than river stage and less sensitive to vertical anisotropy.

- ***Range of Flows***: Minimum flow of 95 gpm occurred for material = Rillito River data, $K_h/K_v = 10$, and river stage = 5 ft. Maximum flow of 2,286 gpm occurred for material = uniform sand, $K_h/K_v = 1$, and river stage = 20 ft.

- ***Hydraulic Conductivity***: For the critical case of 20 ft river stage, total flow into the roadway ranged from 358 gallons per minute (gpm) for K values from Rillito River core data to about 2,286 gpm for K values for uniform sand.

- ***Vertical Anisotropy***: For the critical case of 20 ft river stage and using the lower K value from Rillito River core data, increasing the ratio K_h/K_v from 1 to 10 resulted in a decrease in flow from about 358 to 269 gpm.

- ***River Stage***: For the upper K value from uniform sand, increasing river stage from 5 to 20 feet resulted in an increase in flow from about 812 to 2,286 gpm.

2.3 Case II – Retaining Wall and Roadway as No-Flow Boundaries with Geocomposite Drain

2.3.1 Geometry

The CASE I model for a river distance of 500 ft was modified to simulate the presence of a high permeability geocomposite drain by adding an extra layer of elements to the depressed section. In the vertical cut (retaining wall), the additional elements are 2 ft horizontal by 10 ft vertical. In the horizontal cut (roadbed), the additional elements are 10 ft horizontal by 1 ft vertical.

2.3.2 Boundary Conditions

Boundary conditions were the same as for Case I, with the following exceptions: all nodes in the depressed section were set to no flow, except one roadway element which was set with constant head of –10 feet representing a drain at the intersection of the vertical and horizontal cuts.

2.3.3 Hydraulic Parameters

The elements added to simulate the highly permeable geocomposite drain were assigned a hydraulic conductivity of 0.5 ft/sec with $K_h/K_v = 1$, equivalent to coarse, clean gravel. All other elements in the model were set to hydraulic parameters derived from USGS Rillito River data, or to parameters for the "Uniform Sand" data (GEO-SLOPE Inc., 1998) as in CASE I. Model pressures were compared using a horizontal hydraulic conductivity of 3.3E-4 or 5.15E-5 ft/sec, K_{unsat} function derived from Rillito River data and SEEP/W uniform sand data, and vertical anisotropy ratio $K_h/K_v = 1$ and 10. River stage was simulated at 5 feet and 20 feet.

2.3.4 Results

Under these conditions, the pressures at all nodes beneath the roadbed and retaining wall are virtually zero and the modeled water table is below the cut. Comparing Case I flow in the 500 ft section with Case 2 flow indicates that the presence of the simulated geocomposite resulted in an increase in flow ranging from 1 percent for uniform sand at a river stage of 20 feet, to 9 percent for Rillito data at a river stage of 5 feet.

2.4 Case III – Retaining Wall and Roadway as No-Flow Boundaries without Geocomposite Drain

2.4.1 Geometry

The CASE I model for a river distance of 500 ft used for CASE II was also used for CASE III. However, the geocomposite drain elements were set to have identical hydraulic parameters as all other model elements to evaluate buoyant pressure in the absence of a geocomposite drain.

2.4.2 Boundary Conditions

Boundary conditions were the same as for Case II. All nodes in the depressed section were set to no flow, except the corner element on the roadway near its intersection with the retaining wall, which was set with constant total head of –10 feet representing a drain.

2.4.3 Hydraulic Parameters

All elements in the model were set to the same hydraulic parameters. Model pressures were compared using a horizontal hydraulic conductivity of 3.3E-4 or 5.15E-5 ft/sec, K_{unsat} function derived from Rillito River data and SEEP/W uniform sand data, and vertical anisotropy ratio K_h/K_v = 1,5, and 10. River stage was simulated at 20 feet.

2.4.4 Results

Maximum pressure was observed for the larger values for hydraulic conductivity. Increasing K_h/K_v ratios from 1 to 10 resulted in an increase in pressure beneath the roadbed from about 240 to 590 pounds per square foot (psf), and an increase in pressure behind the retaining wall from about 150 to 400 psf.

3.0 GROUNDWATER QUALITY

The following discussion summarizes available information for environmental site assessments and landfill investigations conducted for areas near the proposed depressed roadway section.. However, no environmental site assessment investigations have been conducted specifically for the properties between the river and the proposed depressed roadway section.

The area on the west bank of the Santa Cruz River along the depressed roadway section has at least three landfills and has been the subject of a number of investigations. The Rio Nuevo South, "A" Mountain, and Nearmont Landfills are shown on Figure 3.1. Rio Nuevo North Landfill (not shown) is located about 2000 feet north of Congress Street, beyond the northern extent of the depressed roadway section. City of Tucson's Environmental Management Division is supervising enhanced bioremediation pilot tests at the Nearmont and "A" Mountain Landfills. The tests include the injection of air and water into the landfills to accelerate the natural degradation process.

Details of landfill construction and investigations at the site were summarized by Korich (2001). The pits were originally excavated for clay by a brick making company, and filled with municipal solid waste during the period from the mid 1950s to the mid 1960s. Depth of the refuse ranges from about 15 to 50 feet. Monitor well locations are shown on Figure 3.1. In 1987, four shallow monitor wells were installed as part of an investigation for a realignment of Mission Road. In 1996, four additional shallow monitor wells were installed as part of an underground storage tank investigation at a former gas station near Congress Street and above ground storage tanks at a former brick manufacturing facility. Six regional aquifer monitor wells were constructed in 1999 (City of Tucson, 2001).

Shallow and regional wells are sampled by City of Tucson's Environmental Management Division on a regularly scheduled basis. Regional aquifer wells are sampled quarterly. Near bioremediation sites, some wells are sampled monthly. Slightly elevated leachate indicators, including chloride and total dissolved solids, occur in some shallow monitor wells. Traces of organic chemicals have been detected in the regional aquifer monitor wells, well within drinking water standards. As of December 2000, chloroform maximum concentration was 6.1 micrograms per liter (μg/l) and tetrachloroethene (PCE) maximum concentration was 0.9 μg/l. The maximum contaminant level for drinking water is 100 μg/l for chloroform and 5 μg/l for PCE.

On the east side of the river, shallow monitor wells WR-248A and WR-249A had trichloroethene (TCE) concentrations of 2.7 and 0.8 μg/L in October 2001. Regional aquifer monitor well WR-271A had a TCE concentration of 4.9 (City of Tucson, personal communication). Maximum contaminant level in drinking water for TCE is 5.0 μg/L. Source of the TCE is believed to be leaking underground storage tanks at the Pioneer Paint site at 438 W. Congress Street.

An environmental site assessment was conducted for City of Tucson, Environmental Management Division in 1995 for property west of the river and south of Congress Street by Environmental Engineering Consultants, Inc. (EEC). For the specific property investigated, the investigation noted possible environmental contamination from the following potential sources:

- Landfills,
- Possible abandoned, but not removed, underground fuel storage tanks at a former gas station on Congress Street , and
- Possible residuals from a dry cleaners. Chemicals used are unknown, but could include PCE and carbon tetrachloride.

The EEC report (1995) noted Underground Storage Tanks (UST) and Leaking Underground Storage Tanks (LUST) listed by the Arizona Department of Environmental Quality at the following locations:

- LUST 292 S. Freeway Current Fuel Tanks (located between the river and the depressed roadway, Figure 3.1)
- LUST 438 W. Congress, 4 hazardous chemical tanks removed
- UST 481 W. Congress, fuel tanks
- UST 1002 W. Congress, two 10,000 gallon fuel tanks
- UST 351 S. Brickyard Lane 1 current fuel tank, 1 current oil tank, 3 removed fuel tanks

From review of aerial photographs, the EEC report (1995) also noted:

- A former gas station at the southwest corner of the Interstate on ramp and Congress Street where a fast food restaurant now exists,
- Former gas stations at 849 and 914 W. Congress
- Auto repair shop at 701 N. Bonilla Street, operated since 1920
- Above ground oil storage tanks (Figure 3.1)

An environmental site assessment has not been conducted for the area between the river and the depressed roadway section. However, the area was discussed in the EEC report

(1995) as follows: *"Various industrial and commercial property lies on the east side of the Santa Cruz River...Most of the structures are motels with minor government offices."* Leaking underground storage tanks were noted in the area between the river and the depressed roadway section and should be further investigated to determine if seepage flow has the potential to mobilize contaminants. Additional environmental site assessment investigations for the specific properties in the area property between the river and the depressed roadway section are recommended.

Previous investigations have indicated numerous potential contaminant sources in the area of the proposed depressed roadway section. During periods of flow in the Santa Cruz River, infiltrated water will produce hydraulic gradients away from the river and contaminants west of the river and east of the roadway should be transported away from the depressed roadway. However, in the period after infiltration from surface flow has stopped, perched groundwater may migrate in different directions, controlled by the topography of the transmissive and perching layers.

Results of groundwater sampling in wells completed in shallow perched groundwater and the regional aquifer indicate that minor impacts from landfill leachate migrating to the shallow perched have been detected and small concentrations of organic chemicals have been detected in the regional aquifer. Current concentrations of contaminants would not likely present a problem for discharge of roadway drainage. However, future activities including bioremediation efforts at landfills and possible creation of sustained flow in the Santa Cruz River for riparian habitat could result in enhanced opportunities for the migration of contaminants. Fortunately, the site has an extensive network of monitor wells and is sampled regularly. Groundwater sampling results should be monitored for increases in contaminant concentrations.

4.0. RETAINING WALLS

It is our understanding that walls will be used to retain the cuts for the depressed section as opposed to laying back the slopes. The walls will range from 13- to 31-feet in height and will be approximately 12,200 ft long with an estimated wall face area of 223,600 ft^2. Furthermore, the available permanent underground easements are greater than 50-feet behind the final wall face. Although permanent underground easements are available it may not be feasible to excavate the soils and build fill or "bottom-up" walls because existing frontage roads, which lie in close proximity to the proposed alignment of the depressed section, will be used to carry diverted I-10 traffic during construction. Therefore, cut walls, placed by a "top-down" construction approach, wherein the wall is either constructed in lifts starting at the ground surface or constructed entirely from the ground surface and subsequently excavated, are preferable to standard reinforced concrete cantilever walls or other fill walls.

Based on the geotechnical information from past ADOT (Arizona Department of Transportation) projects in the area and available permanent underground easements, the two most feasible wall types are soil nail walls and soldier pile walls with or without tiebacks. The features of these walls are briefly described herein. Other alternative retaining wall systems are technically feasible, including micro-pile (root-pile) walls or diaphragm (slurry) walls. Both of these alternative systems offer high structural stiffness (i.e., reduced deflections), but are significantly costlier than the soil nail or the soldier pile wall systems; therefore, these walls are not discussed herein.

4.1 Soil Nail Walls

Soil nail walls were first constructed in the 1970s for temporary support of excavations, but have been increasingly used over the past 25 years in permanent installations. A typical cross-section of a soil nail wall is shown in Figure 4.1. Both ADOT and PCDOT (Pima County Department of Transportation) have successfully used these walls in the past. Examples include the soil nail walls for River Road improvements by PCDOT where heights up to 65-feet in close proximity to settlement-sensitive structures were attained and the embankment retaining walls at I-10 and Congress Street by ADOT.

The soil nail technique is well suited to cut walls, particularly where soil conditions allow cuts to stand vertically for short periods of time and where soils are capable of developing a minimum amount of long term pullout resistance as determined from verification tests on sacrificial nails. There are 6 major steps in the construction of soil nail walls:

1. Excavate a vertical cut to a depth of about 5-feet.
2. Drill holes in the vertical face.
3. Install and grout rebar (nails) as the holes are completed. Specialized equipment is available that drills the hole and installs and grouts the nail simultaneously. The soil nails are generally installed at angles of 15- to 25-degrees below the horizontal.
4. Install drainage geocomposite and wire mesh on the exposed vertical face and apply a 4- to 6-inch shotcrete coating to the vertical face to temporarily resist raveling of the face.
5. Repeat steps 1 to 4 as necessary until the bottom of wall is reached.
6. Construct a permanent precast or cast-in-place fascia.

Soil nails are typically installed in an orthogonal grid pattern with spacing varying from 5ft x 5ft to 6ft x 6ft for the soil conditions anticipated at the project site. Proof tests to verify the design load are performed on a minimum of 5% of the production soil nails installed on each level (bench) or at least one nail if 5% of the total is less than one. Soil nail lengths are generally between 0.7H and 0.9H, where H is the total retained height. For the depths and soil conditions of this project, maximum nail lengths will be approximately 20-feet. Existing utilities behind the wall can be accommodated by splaying the soil nails vertically and horizontally around the utilities.

For costing purposes, a unit cost of $40 per square feet of wall face may be assumed for soil nail walls, excluding the costs for instrumentation that is needed to monitor movements close to settlement-sensitive structures. Assuming a cost of $50,000 for instrumentation, the cost of soil nail walls for the estimated 223,600 ft^2 of wall face along the depressed section will be approximately $9 million. Note that this cost does not include the cost of aesthetic facing. Depending on the type and extent of aesthetic facing, the additional costs may range from $10 to $15 per square feet of wall face or approximately $2.2- to $3.3-million dollars.

4.2 Soldier Pile Walls

Soldier pile walls with and without tiebacks (cantilevered) have been a popular and proven method for cut wall construction for several decades in Arizona and elsewhere.

Cantilever soldier pile walls are cost-effective up to heights of 15- to 18-feet, beyond which tiebacks (ground anchors) are used to prevent excessive deflections as well as obtain a cost-effective wall configuration. A typical configuration of a soldier pile wall with a tieback is shown in Figure 4.2. There are several variations of soldier pile walls ranging from the use of drilled shafts to the driving of steel H-piles. The spacing and depth of soldier piles and need for lagging is typically determined by the capacity of the soldier piles to resist lateral loads through horizontal spanning or "arching" of the soil between shafts. A permanent aesthetic facing is usually attached to the soldier pile wall following construction.

For the soil conditions anticipated at the project site, soldier piles installed in drilled shafts offer the best option. In this case, vertical shafts at a predetermined spacing are drilled from the ground surface along the line of the future wall fascia. A reinforcing cage or an H-pile is lowered into each shaft before the hole is filled with concrete. After construction of all drilled shafts, the soil on the wall side of the shafts is excavated to the full depth of the final wall, and lagging (reinforced concrete or wood), supported on the ends by adjacent shafts, is placed against the exposed soil face as the excavation proceeds. The lagging is used to retain the soil face and transmit the lateral pressures to the solider piles. The installation of lagging is possible only if the soil is able to arch the horizontal distance between shafts. Therefore, selection of an appropriate spacing of the shafts is very critical to the success of this method.

Drilled shaft diameters for this project could conceivably vary between 24 and 36 inches with spacing varying between 8 and 12 feet, depending on soil properties and the height of retained earth. Either the spacing of soldier piles may be reduced to 4- to 6-feet or a single row of tiebacks may be installed for wall heights greater approximately 15-feet or locations where lateral deflections are more than permissible. Tiebacks will require an underground easement of up to 35-feet. Existing utilities behind the wall can be accommodated by splaying the tiebacks vertically and horizontally around the utilities. Several tests are conducted on tiebacks to verify their capacity. These tests include pre-production tests to verify the design load safety factor, performance tests on 2% to 10% of tiebacks to verify the long-term performance of the tieback under the design load, proof tests on each tieback to determine the behavior of tieback after installation and lift-off tests on each tieback to confirm the load in the tieback after the completion of tieback and lock-off of the applied load.

For costing purposes, a unit cost of \$60 per square feet of exposed wall face may be assumed for soldier pile walls, excluding the costs for instrumentation that is needed to monitor movements close to settlement-sensitive structures. Assuming a cost of \$50,000 for instrumentation, the cost of soldier pile walls for the estimated 223,600 ft^2 of wall face along the depressed section will be approximately \$13.5 million. Note that this cost does not include the cost of aesthetic facing. Depending on the type and extent of aesthetic facing, the additional costs may range from \$10 to \$15 per square feet of wall face or approximately \$2.2- to \$3.3-million dollars.

4.3 Wall Drainage

Regardless of the wall type, it is critical that adequate measures to prevent build-up of hydrostatic pressure behind the wall are implemented. Drainage of water from behind soil nail or soldier pile lagging walls is routinely implemented by placing a geocomposite drain (a dimpled sheet of plastic with a geotextile filter cover layer) against the back face (i.e., soil side) of the wall. In the case of soil nail walls the geocomposite drain is placed against the exposed soil face prior to the installation of reinforcing wire mesh and shotcreting. For soldier pile lagging walls, the geocomposite drain is placed against the exposed soil face prior to installation of the lagging. The geocomposite drain collects the seepage and channels it towards the bottom of the wall where "weep holes" or collector pipes are provided to lead the water away from the wall face. Typical configurations of the wall drainage for soil nail and soldier pile walls are shown in Figures 4.1 and 4.2, respectively; examples of wall drain details are shown in Figure 4.3.

Geocomposite drains are typically 2-feet wide and are generally installed at 5-feet on centers. For additional conservatism, geocomposites may be considered for the entire exposed face of wall. Geocomposites are widely available through many manufacturers (e.g., Tensar, Mirafi, Tenax, Amoco, Ameridrain, etc.[1]) and in sizes that can easily accommodate the anticipated flows from the Santa Cruz River. Costs for geocomposite drains are included in the unit costs reported for the soil nail and soldier pile walls.

[1] URS does not endorse any proprietary products or manufacturers. Trade or manufacturer's names appear herein only as examples. There are several manufacturers that carry similar products.

5.0 PAVEMENTS

Both asphaltic concrete pavements (ACP) and Portland cement concrete pavements (PCCP) are used along the existing I-10 corridor. Since the existing I-10 is at-grade no special provisions have been made to mitigate the effects of possible buoyancy or build-up of hydrostatic pressures such as those that would occur along the proposed depressed section if adequate subsurface drainage measures were not implemented. The buoyancy forces can be mitigated by either designing the pavement structure to resist uplift forces (e.g., provide tie downs connected to PCCP or place the pavement structure on a heavy platform) or designing an adequate pavement drainage system to dissipate water pressures, collect the water under the roadway and drain it in to the drainage system for the depressed section.

Pavements are commonly designed to provide drainage for water seeping down from the surface because added moisture in aggregate base and subbase can result in a loss of pavement stiffness on the order of 50 percent or more (AASHTO, 1993). It is a standard practice with ADOT, PCDOT and COT to design pavement systems with adequate drainage. A typical drainable pavement system is shown in Figure 5.1. The permeable base shown in Figure 5.1 is recommended to have a minimum permeability of 1000 ft/day. This permeability will allow for drainage of the pavement within a few hours; i.e., a condition that qualifies as "excellent drainage" as defined by AASHTO (1993). Figure 5.2 shows the relationship of permeability for free-draining or open-graded aggregate base (OGAB) as compared to denser-graded aggregate base (DGAB).

The above drainage measures are used to collect and drain water that is seeping down from the road surface only. For the depressed section, not only must this condition be addressed but the roadway must also be designed to capture the bottom-up seepage flows from the Santa Cruz River and relieve the buoyancy forces should the proposed drainage disposal system (pumps, vaults, etc.) malfunction. This can be achieved by a number of methods including increasing the thickness of the OGAB to account for increased flows from the bottom-up or by the use of a geocomposite underdrain system specifically designed for that purpose.

In order to provide optimum drainage, the outflow capacity of the drainage layer must be sufficient to drain the pavement section within a few hours of a moisture event. A conventional 4-inch-thick OGAB has proven adequate to meet this drainage requirement as this layer typically has a permeability of at least 1000 ft/day. Therefore, a 4-inch thick-free draining base layer has a transmissivity (i.e., permeability multiplied by the thickness) of about 300 ft^2/day. For a typical roadway gradient of 0.02 (a 2 percent grade), the OGAB has a flow capacity between 6 ft^3/day (0.00007 ft^3/sec) per ft length of road. The depressed section is approximately 6,000 ft long and the anticipated maximum flow rate is 5.1 ft^3/sec. Assuming typical values of 1000 ft/day for permeability of an OGAB and a roadway gradient of 0.02, a 4-inch thick OGAB layer would be required.

A major concern with the use of OGAB to trap upward flowing water is that it may become contaminated with fines (particle sizes finer than the No.200 sieve). Based on hydraulic conductivity tests, AASHTO (1993) notes a decrease in permeability from 10 ft/day with 0 percent fines down to 0.07 ft/day with addition of only 5 percent non-plastic fines and 0.001 ft/day with 10 percent non-plastic fines. An additional order of magnitude decrease was observed with base containing plastic fines. Thus, filters or a geotextile fabric must be provided to prevent migration of fines into the OGAB.

A potential alternative for both improved drainage and mitigation of buoyancy forces would be to incorporate a low compression, geocomposite drainage layer tied into the roadway edge drains as shown in Figure 5.3. An upper and lower geotextile layer can be provided to prevent migration of fines with water seepage from both below and above the roadway surface. An example detail of drainage with geocomposite and edge drains is shown in Figure 5.4. The edge drains can be integrated into the overall drainage system for the depressed section.

The geocomposite underdrain must have a flow capacity that can rapidly drain the pavement section so as to prevent saturation of the base as well as build-up of pore pressure. To prevent build-up of pore pressure it is critical that an air-void (or atmospheric condition) exists within the geocomposite underdrain to provide a capillary break and ensure continuous flow of collected water. There are several proprietary products that are available on the market that can provide the required transmissivity and flow characteristics (e.g., Tendrain manufactured by Tenax). An added advantage of the geocomposite underdrain is its tensile capacity that can lead to an increase in structural stiffness of the pavement section. ADOT allows an increase of R-value by 10 if a geosynthetic layer is used. Thus, a geocomposite underdrain will not only take care of the subsurface drainage issues but also potentially result in a reduced pavement thickness.

Regardless, of whether an OGAB or a geosynthetic is used, it is very important that the drainage system be carefully designed and specified to prevent build-up of pore pressures. Since the overall pavement section can be reduced because of the stiffness provided by the geocomposite underdrain, the initial greater cost of the geocomposite system is offset the savings in pavement section. In our experience, the costs of the two options are expected to be similar with the geocomposite option probably being less expensive. For costing purposes, it is recommended that a unit cost of $40/yd^2 of pavement be used based on assumption of OGAB.

6.0 REFERENCES

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Weatherby, D. E. (1998). Design Manual for Permanent Ground Anchor Walls, FHWA Report No. FHWA-RD-97-130, McLean, VA.

FIGURES

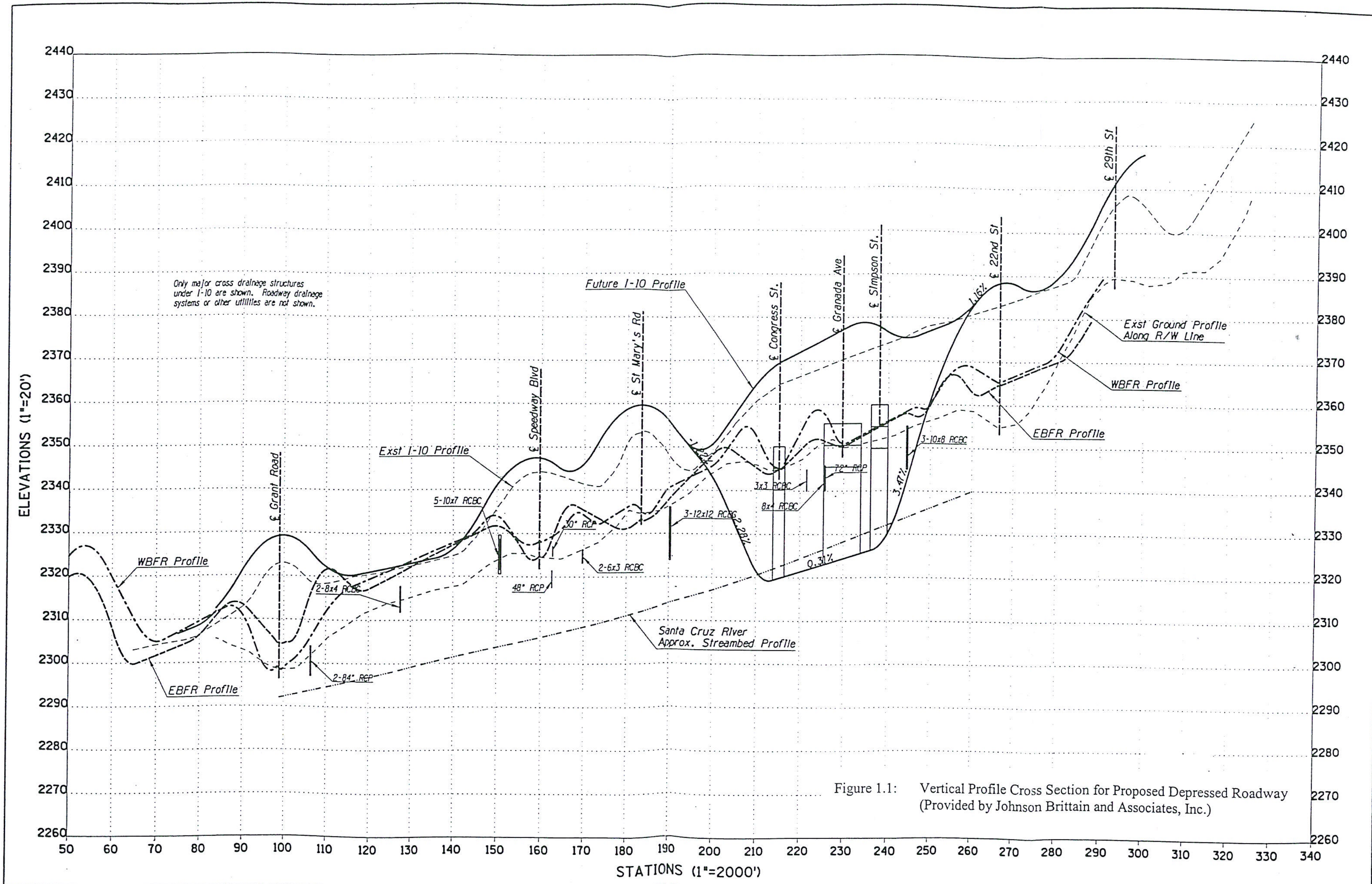


Figure 1.1: Vertical Profile Cross Section for Proposed Depressed Roadway (Provided by Johnson Brittain and Associates, Inc.)

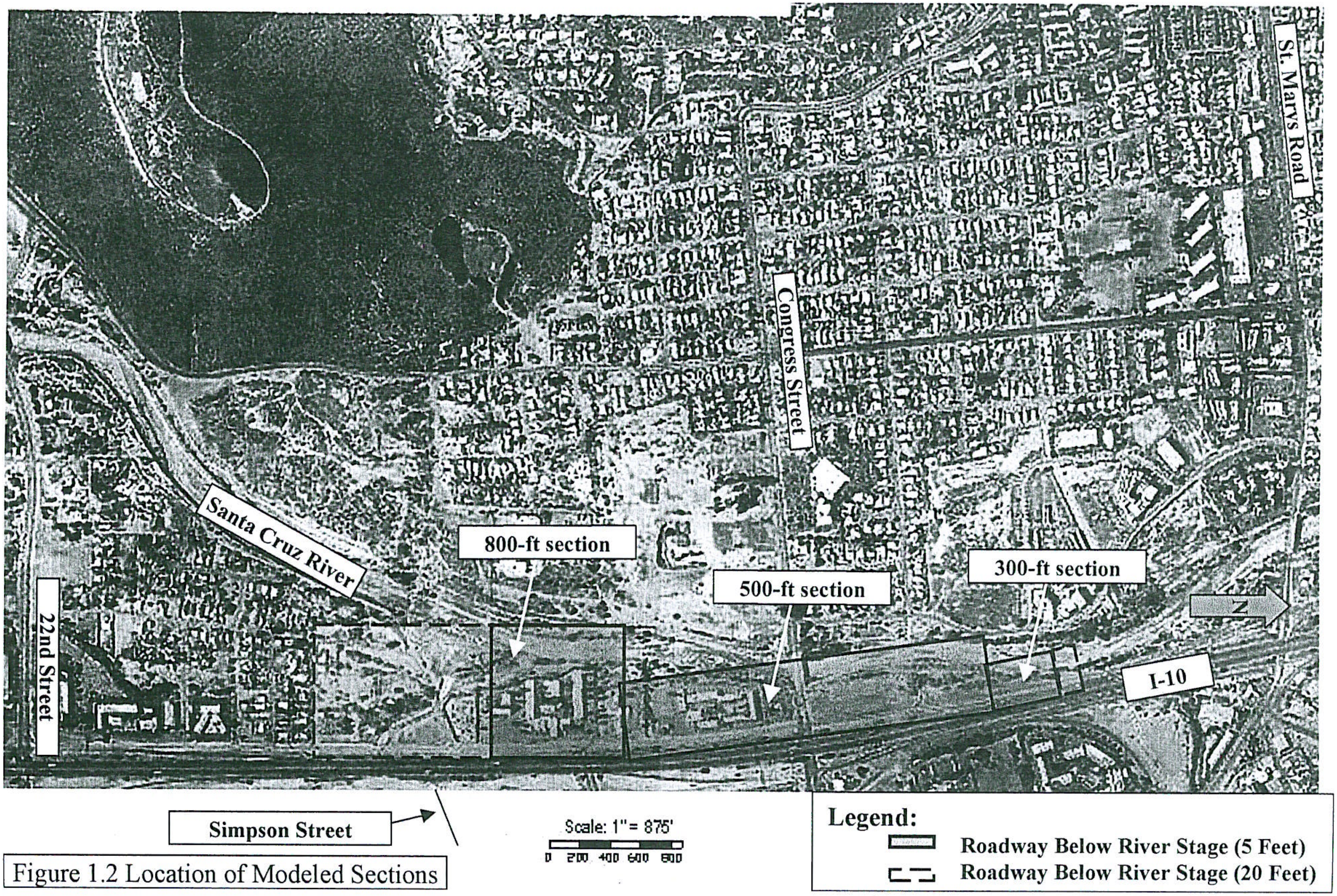
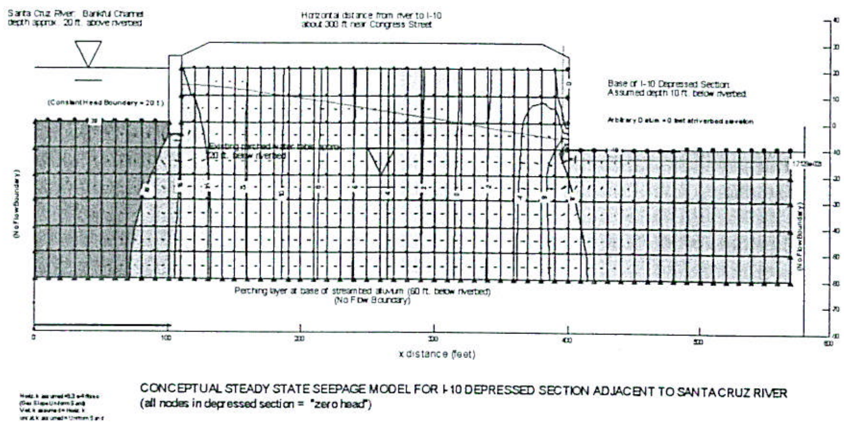
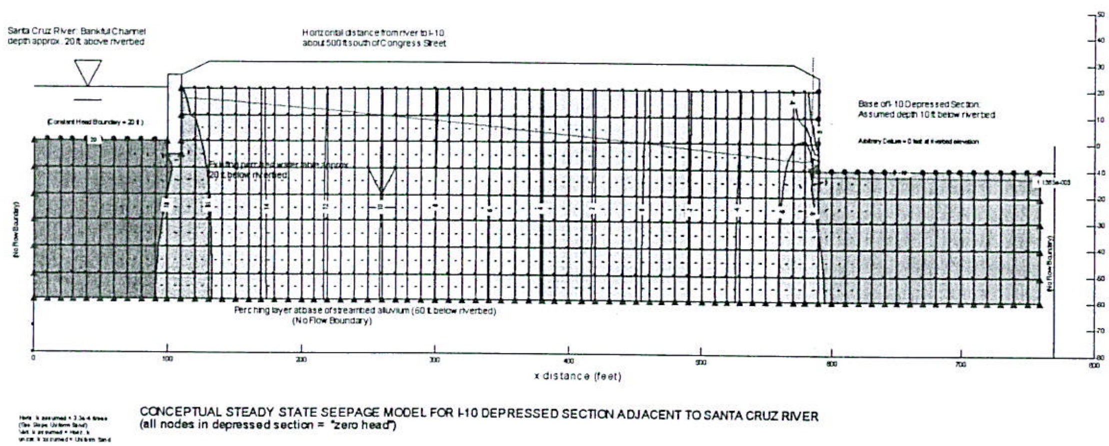


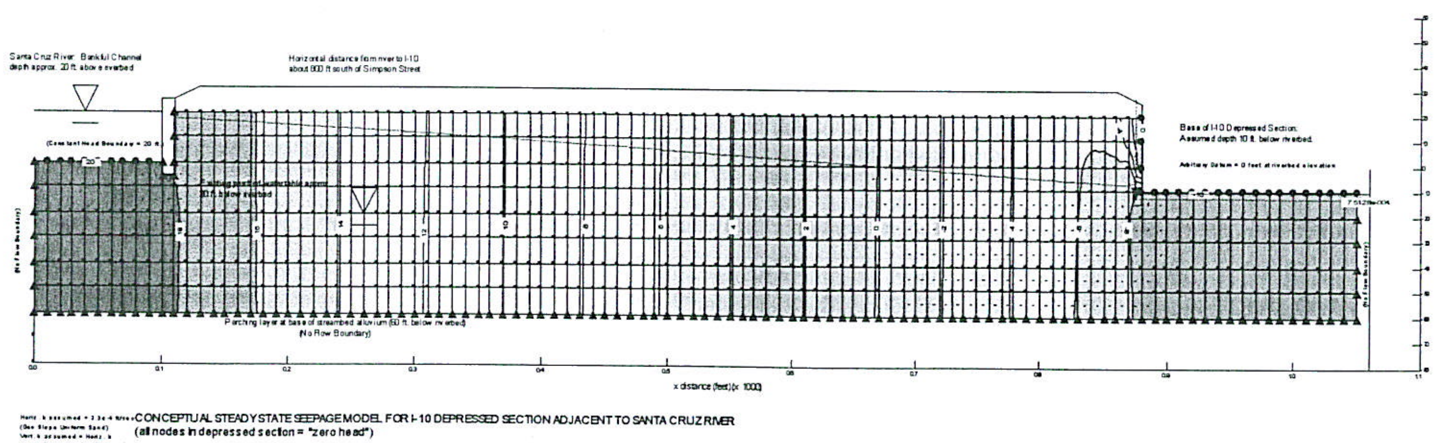
Figure 1.2 Location of Modeled Sections



300 feet

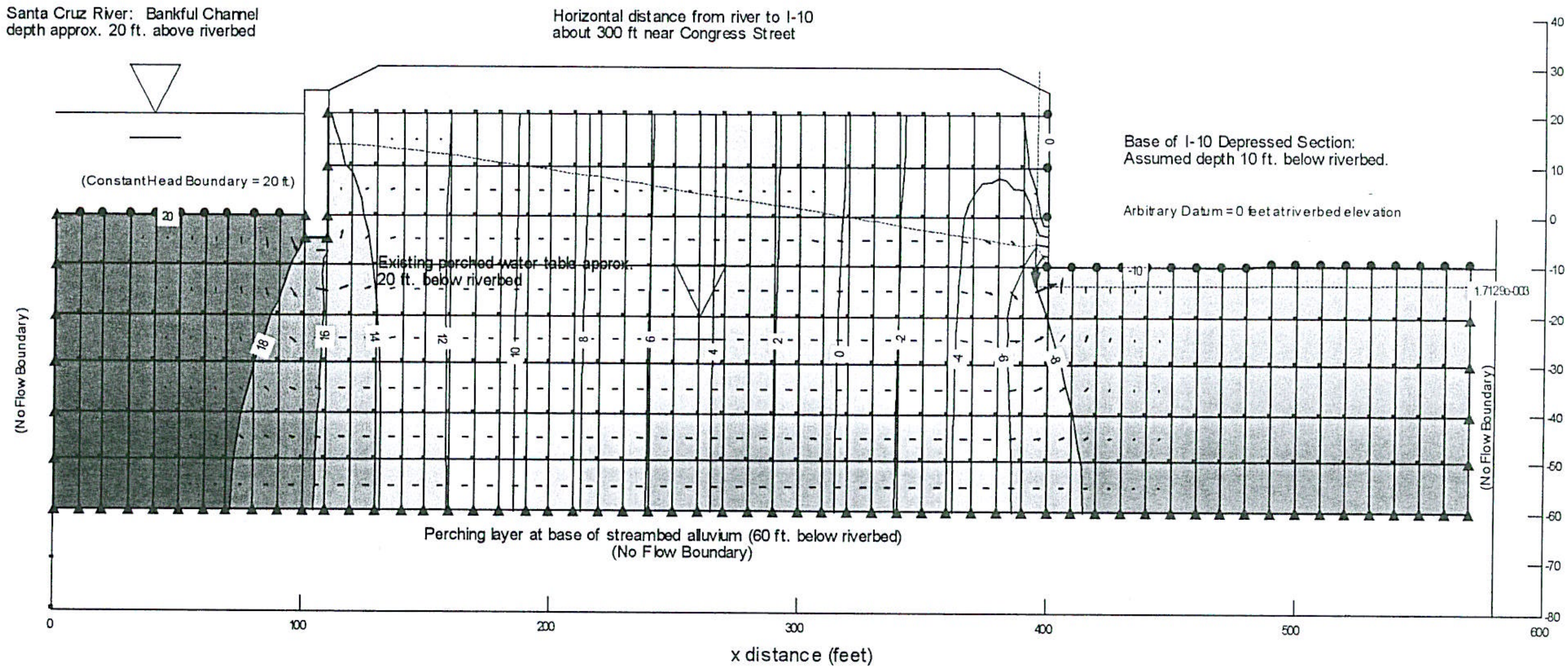


500 feet



800 feet

Figure 2.1 - Modeled sections for roadway at 300, 500, and 800 feet from the Santa Cruz River.



Horiz. k assumed =3.3 e-4 ft/sec
(Geo Slope Uniform Sand)
Vert. k assumed = Horiz. k
unsat. k assumed = Uniform Sand

CONCEPTUAL STEADY STATE SEEPAGE MODEL FOR I-10 DEPRESSED SECTION ADJACENT TO SANTA CRUZ RIVER
(all nodes in depressed section = "zero head")

Figure 2.2 – Boundary Conditions for Steady State Flow Simulations.

Figure 2.3 - River Stage at Peak Flow, Santa Cruz River at Tucson

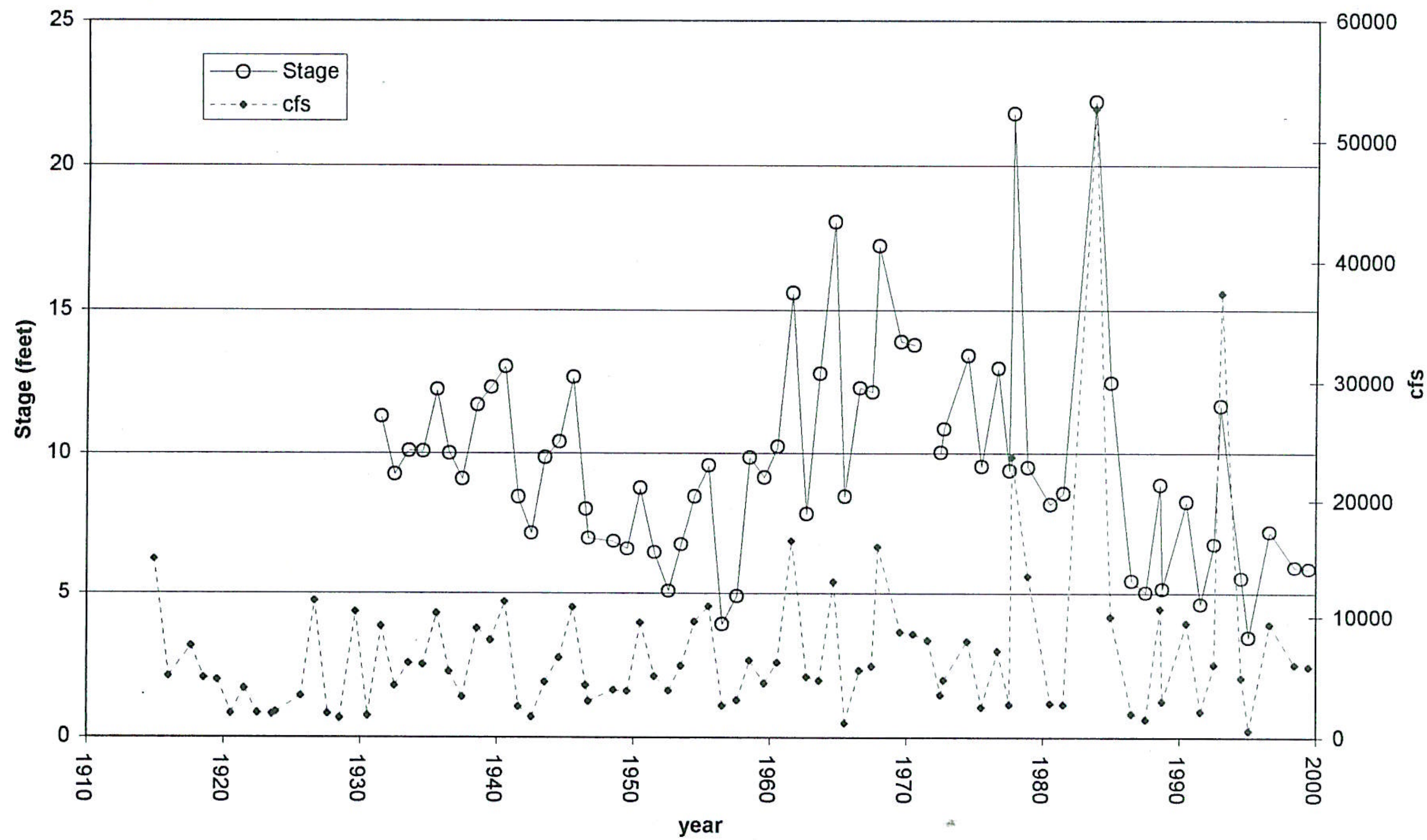
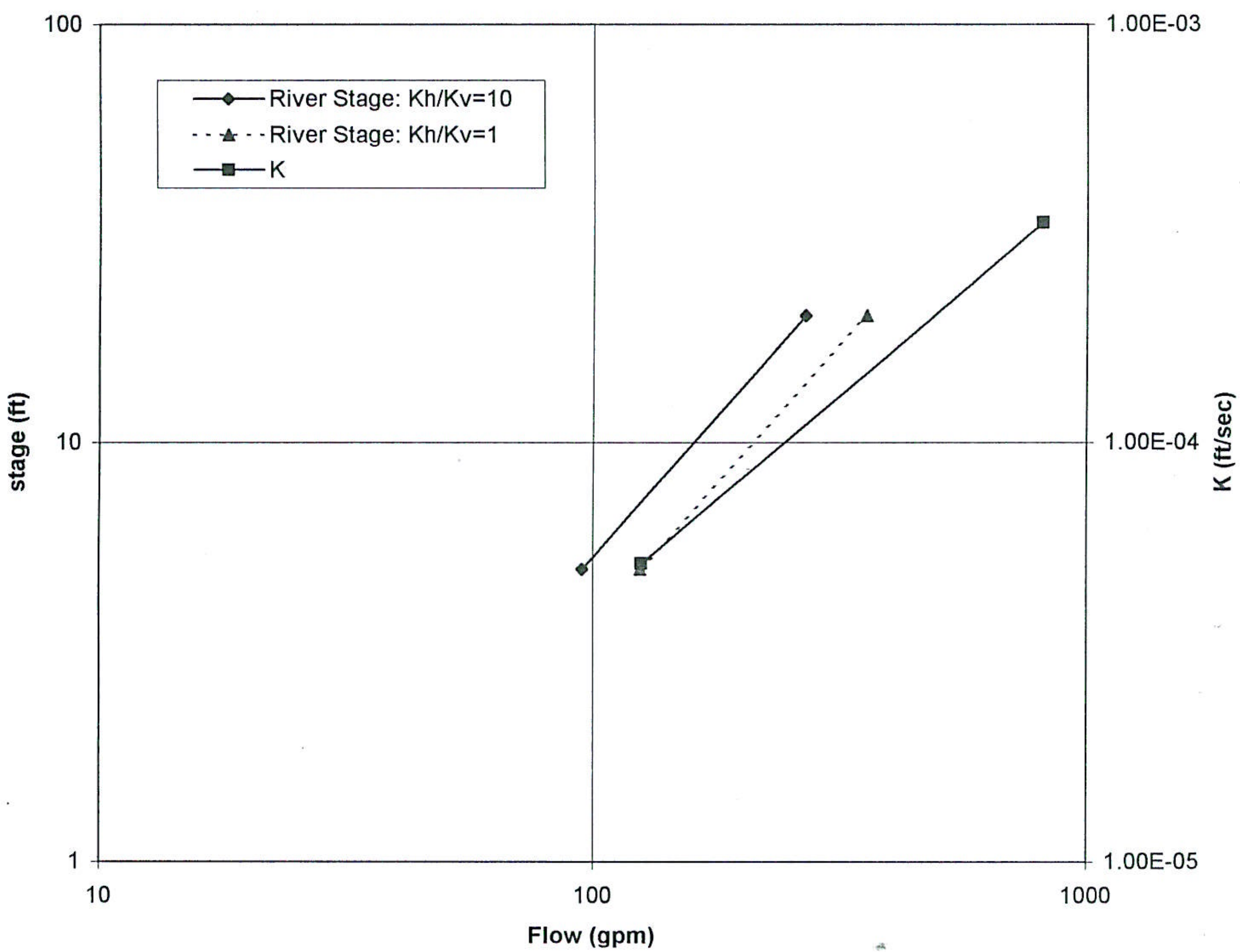
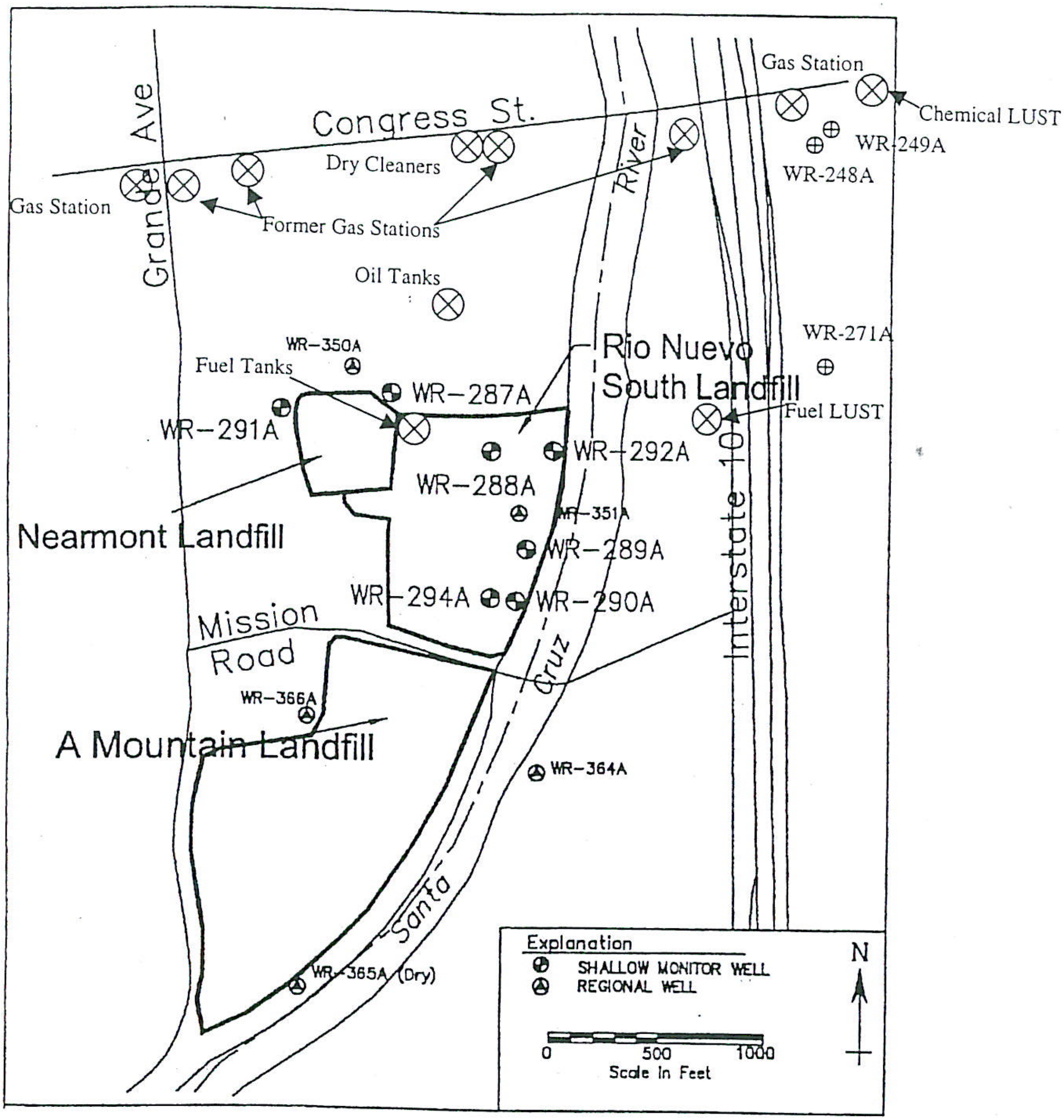


Figure 2.4 - Case I: Model Sensitivity to Input Parameters





(modified from Korich, 2000)

Figure 3.1: Site Map Showing Potential Sources of Contamination

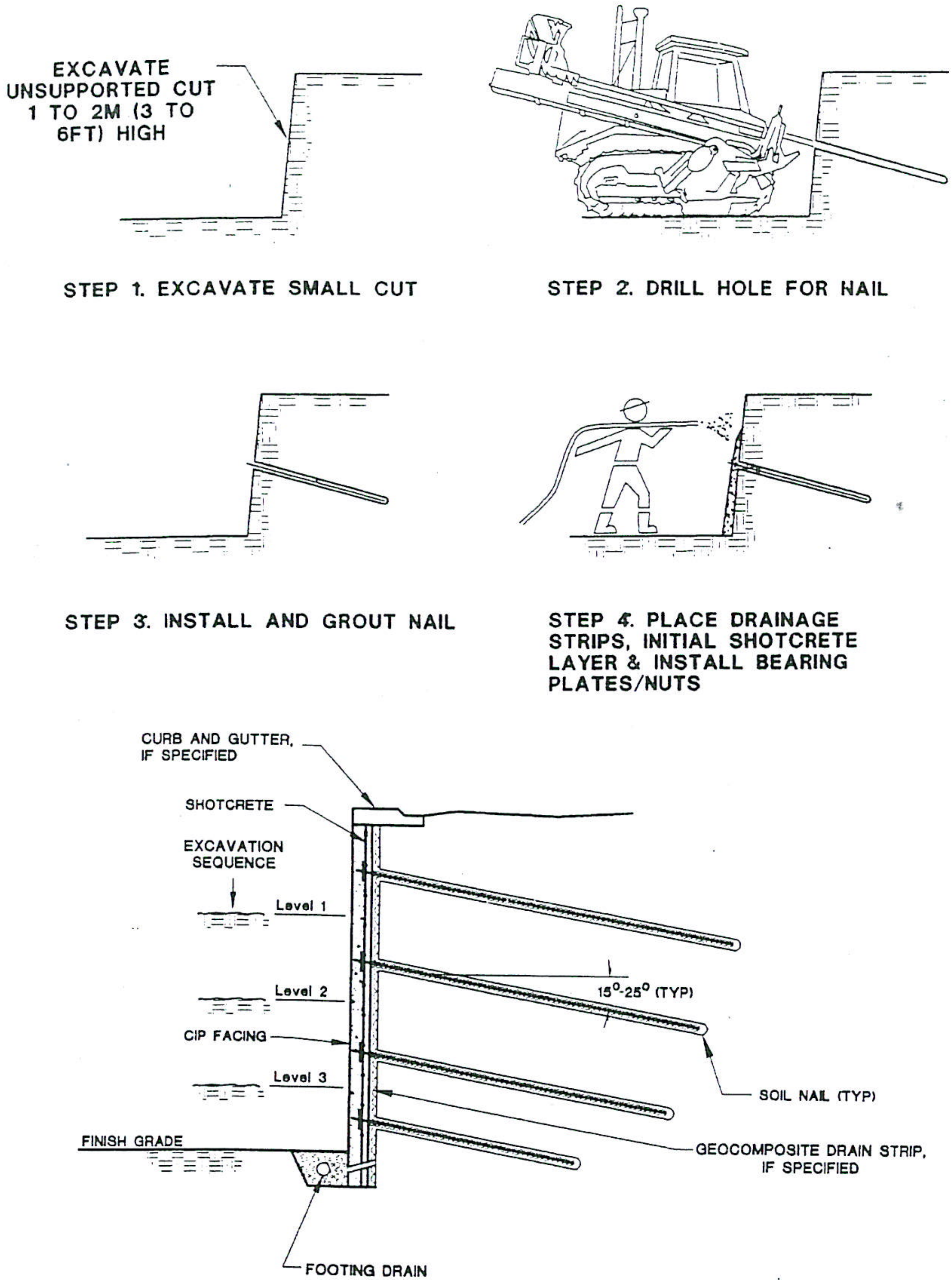


Figure 4.1: Typical Steps in Soil Nail Wall Construction and a Typical Section Through a Vertical Wall (Porterfield, *et al.*, 1994)

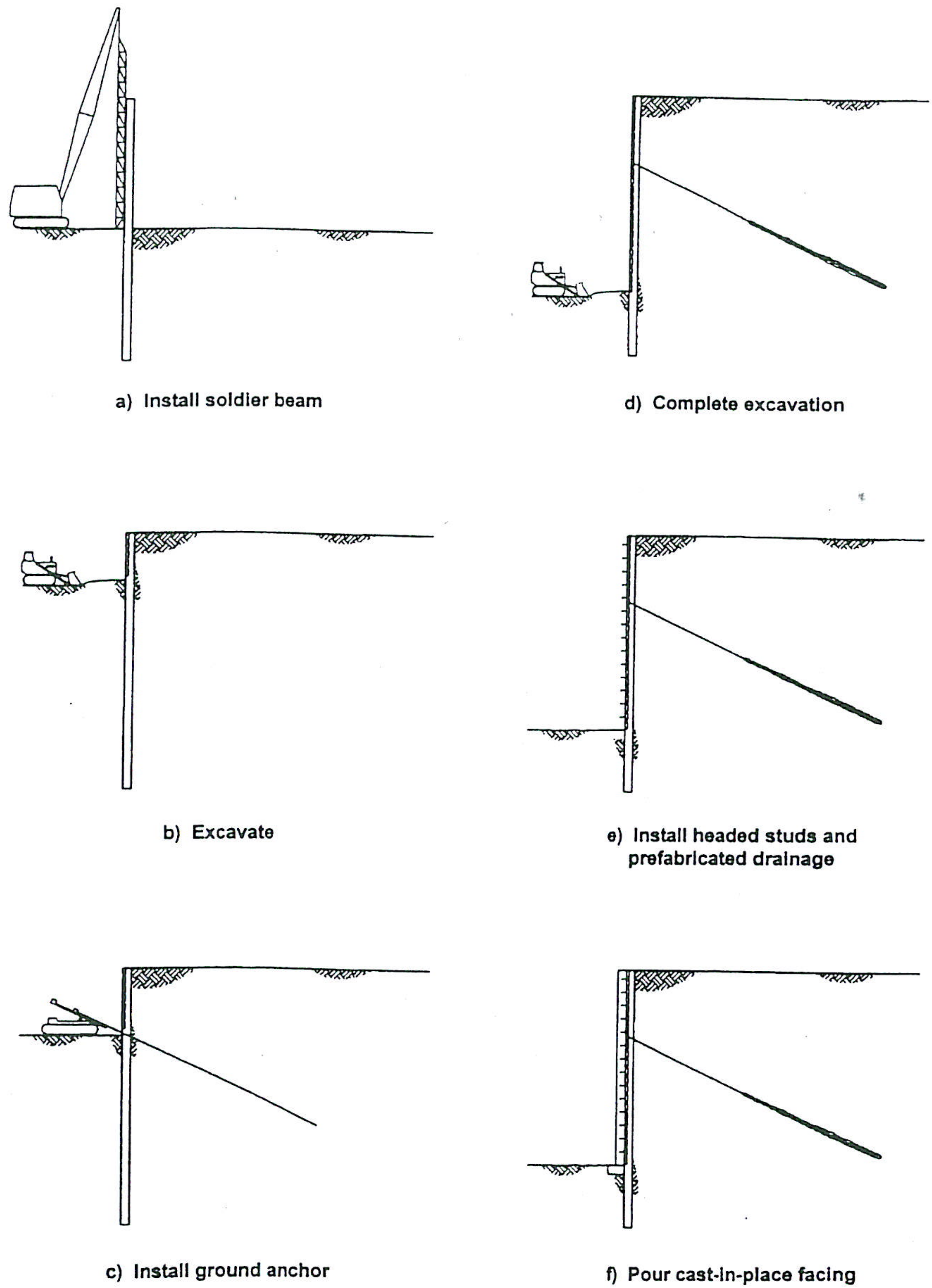
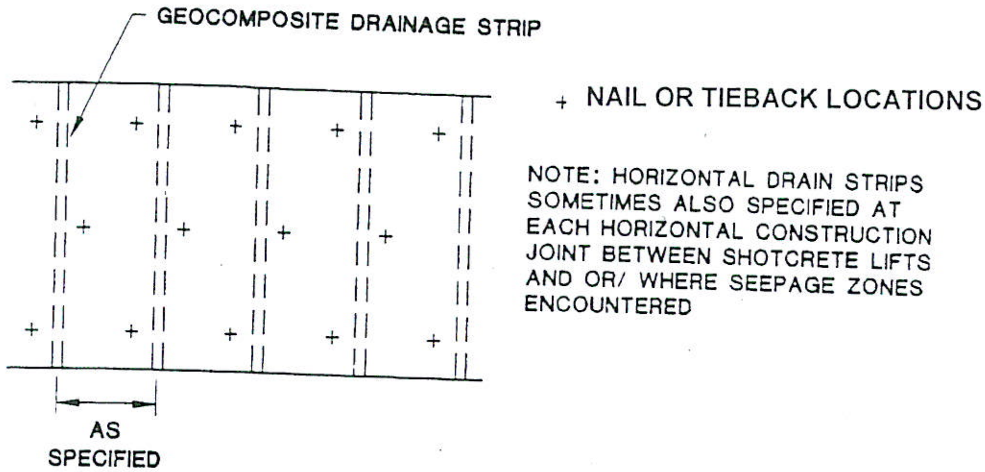
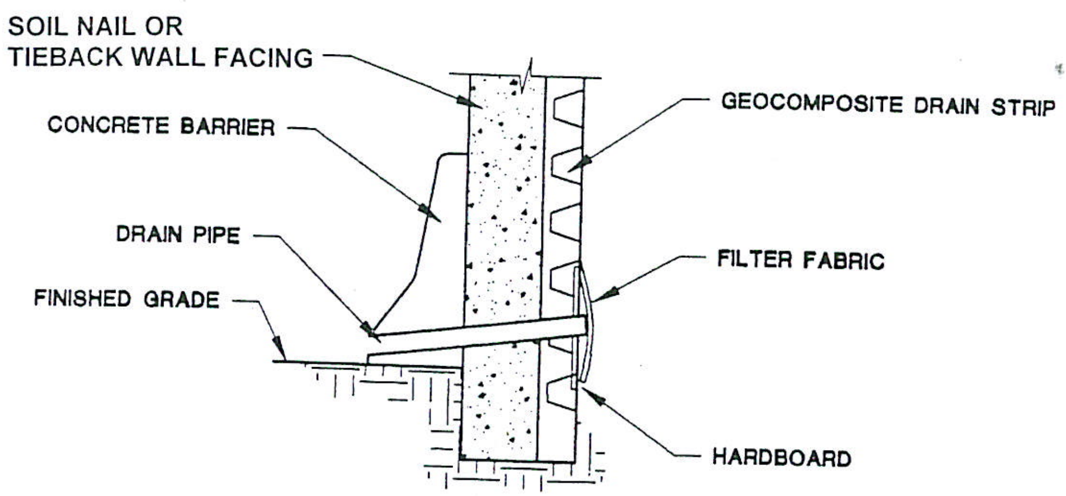


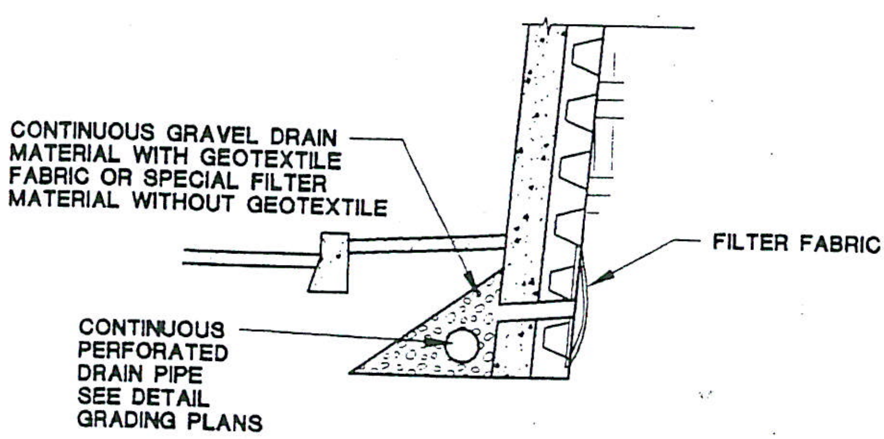
Figure 4.2: Typical Steps in Soldier Pile Wall (with Tieback) Construction (Weatherby, 1998)



GEOCOMPOSITE DRAIN STRIPS



TOE DRAIN OUTLET PIPE



FOOTING DRAIN

Figure 4.3: Examples of Wall Drain Details (Porterfield, *et al.*, 1994)

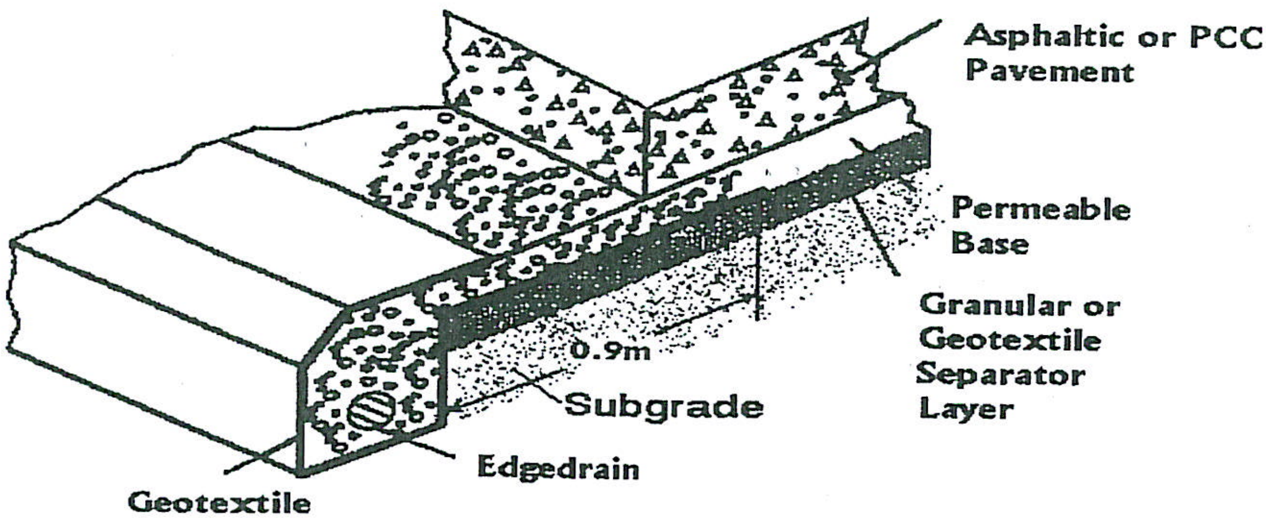


Figure 5.1: Drainable Pavement System (Christopher, 2001)

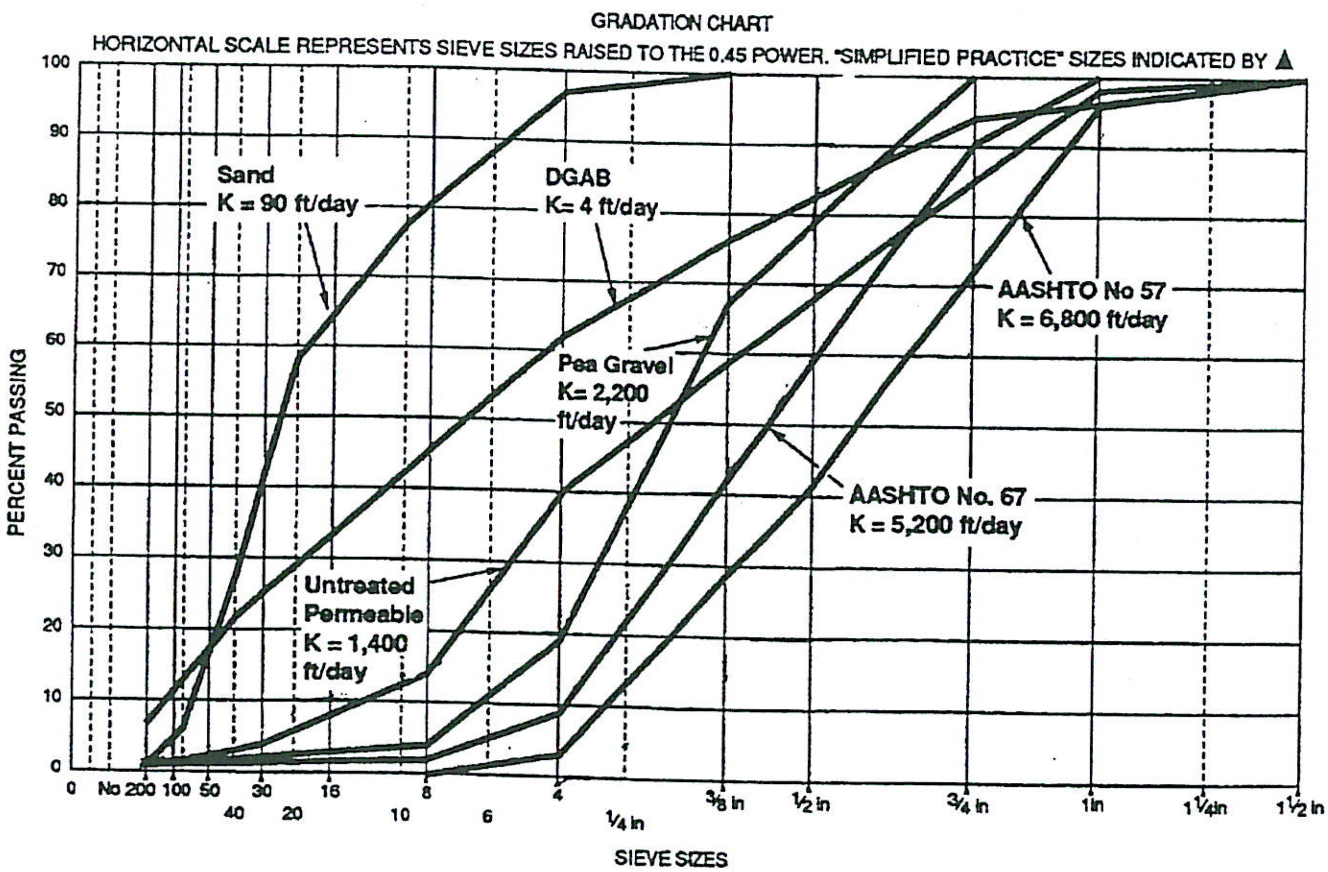


Figure 5.2: Gradation and Permeability of Base Course Materials (Christopher, 2001)

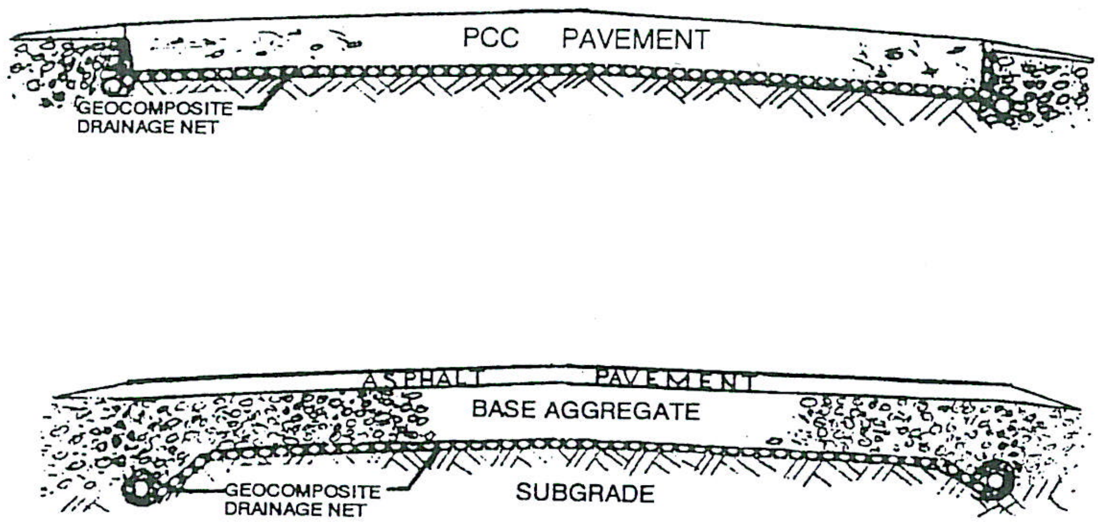


Figure 5.3: Some Potential Configurations for Use of Geocomposite Drainage Layers (Christopher, 2001)

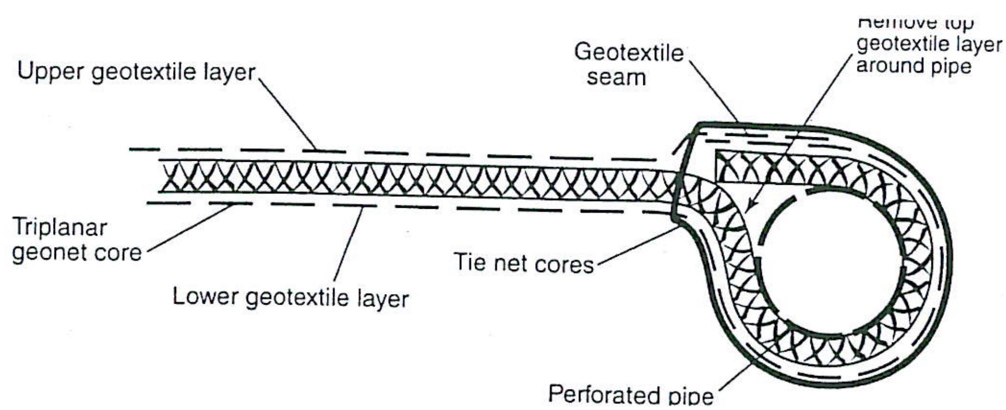


Figure 5.4: Example Edge Drain Construction with Geocomposite Drains (Christopher, 2001)

Appendix C
Cost Estimates

FUNDING REQUIRED FOR
Interstate 10, St Marys to 22nd St

		PROPOSED PROJECT	ORIGINAL PROJECT	CONSTRUCT DEPRESSION AFTER MAINLINE WIDENING
PROJECT COSTS		$76.5 million	$25.9 million	$78.3 million
FUNDING				
	ADOT	$25.9 million	$25.9 million	$25.9 million
	COT	$50.6 million	$0	$78.3 million
TOTAL COSTS		$76.5 million	$25.9 million	$104.2 million

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SUMMARY OF COSTS
I10 Depression, St Marys to 22nd St

ITEM	PROPOSED PROJECT COST		ORIGINAL PROJECT COST		CONSTRUCT DEPRESSION AFTER MAINLINE WIDENING COST	
REMOVALS		$2,439,969		$1,436,712		$2,955,136
ROADWAY		$16,434,760		$6,721,670		$17,104,760
STRUCTURES		$20,124,098		$6,713,140		$20,124,098
ONSITE DRAINAGE		$4,794,300		$799,800		$4,794,300
CROSS DRAINAGE		$2,772,004		$210,936		$2,772,004
UTILITY RELOCATION		$1,000,000		$200,000		$1,000,000
MISCELLANEOUS		$1,446,000		$946,000		$1,446,000
LIGHTING & SIGNALS		$710,000		$215,000		$710,000
SUBTOTAL	$49,721,131	$49,721,131	$17,243,257	$17,243,257	$50,906,298	$50,906,298
TRAFFIC CONTROL @ 5%	$2,474,057		$862,163		$2,533,315	
CONTINGENCIES @ 15%	$7,458,170		$2,586,489		$7,635,945	
CONSTRUCTION ADMINISTRATION @ 15%	$7,458,170		$2,586,489		$7,635,945	
DESIGN @ 15%	$7,458,170		$2,586,489		$7,635,945	
TOTAL	$74,569,697		$25,864,886		$76,347,448	
ABUTMENT FOR FUTURE BRIDGE FROM PEDESTRIAN CROSSING TO CLARK	$1,920,000				$1,920,000	
TOTAL	$76,489,697		$25,864,886		$78,267,448	

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DETAILED COST ESTIMATE
I10 Depression, St Marys to 22nd St

ITEM	UNIT PRICE	PROPOSED PROJECT QUANTITY	PROPOSED PROJECT COST		ORIGINAL PROJECT QUANTITY	ORIGINAL PROJECT COST		CONSTRUCT DEPRESSION AFTER MAINLINE WIDENING QUANTITY	CONSTRUCT DEPRESSION AFTER MAINLINE WIDENING COST	
REMOVALS				$2,439,969			$1,436,712			$2,955,136
REMOVE EXST PVMT	$8 /sy	148,007 sy	$1,184,055		96,996 sy	$775,968		194,664 sy	$1,557,312	
REMOVE EXST WALL	$80 /lf	3,900 lf	$312,000		950 lf	$76,000		4,950 lf	$396,000	
REMOVE EXST NOISE WALL (ebfr)	$80 /lf	280 lf	$22,400					280 lf	$22,400	
REMOVE EXST BRIDGES	$100,000 /ea	2 ea	$200,000		2 ea	$200,000		2 ea	$200,000	
REMOVE EXST PEDESTRIAN TUNNEL	$150 /cy	324 cy	$48,660		324 cy	$48,600		444 cy	$66,570	
REMOVE EXST MEDIAN BARRIER	$10 /lf	7,050 lf	$70,500		7,050 lf	$70,500		7,050 lf	$70,500	
REMOVE EXST BARRIER	$10 /lf	7,000 lf	$70,000		5,400 lf	$54,000		11,000 lf	$110,000	
REMOVE EXST CURB/GUARDRAIL	$3 /lf	28,730 lf	$86,190		13,600 lf	$40,800		28,730 lf	$86,190	
REMOVE EXST MAINLINE PVMT DRAIN SYSTEM	$15 /lf	6,450 lf	$96,750		6,450 lf	$96,750		6,450 lf	$96,750	
REMOVE EXST 36" RCP (Franklin)	$15 /lf	350 lf	$5,250		230 lf	$3,450		350 lf	$5,250	
REMOVE EXST 36" RCP (Alameda)	$15 /lf	400 lf	$6,000		250 lf	$3,750		400 lf	$6,000	
REMOVE EXST 42" RCP (Congress)	$15 /lf	500 lf	$7,500		500 lf	$7,500		500 lf	$7,500	
REMOVE EXST 1-3' x 3' RCBC (TCC Wash)	$150 /cy	200 cy	$30,000		125 cy	$18,750		200 cy	$30,000	
REMOVE EXST 1-8' x 4' RCBC (Simpson Wash)	$150 /cy	502 cy	$75,364		271 cy	$40,644		502 cy	$75,364	
REMOVE EXST 72" RCP (Simpson Wash)	$30 /lf	450 lf	$13,500					450 lf	$13,500	
REMOVE EXST 3-10' x 8' x 400' RCBC (18th Street Wash)	$150 /cy	1,412 cy	$211,800					1,412 cy	$211,800	
ROADWAY				$16,434,760			$6,721,670			$17,104,760
EXCAVATION	$5 /cy	1,740,000 cy	$8,700,000					1,874,000 cy	$9,370,000	
BORROW	$5 /cy				134,000 cy	$670,000				
PCCP	$35 /sy	194,664 sy	$6,813,240		143,653 sy	$5,027,859		194,664 sy	$6,813,240	
AGGREGATE BASE(6")	$20 /cy	8,111 cy	$162,220		5,986 cy	$119,711		8,111 cy	$162,220	
CONCRETE MEDIAN BARRIER	$50 /lf	7,050 lf	$352,500		7,050 lf	$352,500		7,050 lf	$352,500	
CONCRETE BARRIER	$40 /lf	5,090 lf	$203,600		11,000 lf	$440,000		5,090 lf	$203,600	
CONCRETE CURB	$8 /lf	25,400 lf	$203,200		13,950 lf	$111,600		25,400 lf	$203,200	
STRUCTURES				$20,124,098			$6,713,140			$20,124,098
NEW RETAINING WALL	$55 /sf	223,600 sf	$12,298,000		40,000 sf	$2,200,000		223,600 sf	$12,298,000	
NEW BRIDGES										
I10 OVER CONGRESS ST	$100 /sf				31,200 sf	$3,120,000				
I10 OVER MISSION LANE	$100 /sf				12,600 sf	$1,260,000				

CONGRESS ST OVER I10	$75 /sf	30,982 sf	$2,323,650					30,982 sf	$2,323,650	
PEDESTRIAN BRIDGE OVER I10	$120 /sf	15,219 sf	$1,826,280					15,219 sf	$1,826,280	
CLARK ST (GRANADA) OVER I10	$75 /sf	20,015 sf	$1,501,125					20,015 sf	$1,501,125	
SIMPSON ST (MISSION LANE) OVER I10	$75 /sf	12,581 sf	$943,575					12,581 sf	$943,575	
PLANTERS ON BRIDGES										
PEDESTRIAN BRIDGE OVER I10	$120 /sf	6,809 sf	$817,080					6,809 sf	$817,080	
CLARK ST (GRANADA) OVER I10	$97.92 /sf	4,232 sf	$414,388					4,232 sf	$414,388	
NEW CONCRETE PEDESTRIAN TUNNEL (1-12'x12'x200')	$300 /cy				444 cy	$133,140				
ONSITE DRAINAGE				**$4,794,300**			**$799,800**			**$4,794,300**
PUMP STATION (10 cfs)	$500,000 /ls	1 ls	$500,000					1 ls	$500,000	
STORAGE BASIN (17 acre-ft)	$300 /cy	7,200 cy	$2,160,000					7,200 cy	$2,160,000	
BASIN EXCAVATION	$5 /cy	50,000 cy	$250,000					50,000 cy	$250,000	
24" RCP (Drain to Pump)	$60 /lf	500 lf	$30,000					500 lf	$30,000	
MEDIAN CATCH BASIN	$5,000 /ea	30 ea	$150,000		17 ea	$85,000		30 ea	$150,000	
FREEWAY CATCH BASIN	$5,000 /ea	64 ea	$320,000		49 ea	$245,000		64 ea	$320,000	
MANHOLES	$5,000 /ea	30 ea	$150,000		15 ea	$75,000		30 ea	$150,000	
24" RCP	$60 /lf	7,380 lf	$442,800		6,580 lf	$394,800		7,380 lf	$442,800	
30" RCP	$65 /lf	1,100 lf	$71,500					1,100 lf	$71,500	
36" RCP	$80 /lf	1,150 lf	$92,000					1,150 lf	$92,000	
48" RCP	$115 /lf	400 lf	$46,000					400 lf	$46,000	
54" RCP	$120 /lf	500 lf	$60,000					500 lf	$60,000	
REMOVE & REPLACE EXST FRONTAGE ROAD PAVEMENT DRAINAGE SYSTEM	$120 /lf	4,350 lf	$522,000					4,350 lf	$522,000	
CROSS DRAINAGE				**$2,772,004**			**$210,936**			**$2,772,004**
42" RCP (Franklin)	$105 /lf	400 lf	$42,000		220 lf	$23,100		400 lf	$42,000	
36" RCP (Alameda)	$80 /lf	2,700 lf	$216,000		250 lf	$20,000		2,700 lf	$216,000	
42" RCP (Congress)	$100 /lf	1,000 lf	$100,000		500 lf	$50,000		1,000 lf	$100,000	
42" RCP (TCC Wash)	$100 /lf				230 lf	$23,000				
60" RCP (TCC Wash)	$160 /lf	400 lf	$64,000					400 lf	$64,000	
1-8' x 4' x 247' RCBC (Simpson Wash)	$350 /cy				271 cy	$94,836				
2-10' x 6' x 450' RCBC (Simpson Wash)	$300 /cy	1,022 cy	$306,450					1,022 cy	$306,450	
6-10' x 8' x 400' RCBC (18th Street Wash)	$300 /cy	2,679 cy	$803,640					2,679 cy	$803,640	
PUMP STATION (10 cfs)	$500,000 /ls	1 ls	$500,000					1 ls	$500,000	
24" RCP (Drain to Pump)	$70 /lf	3,600 lf	$252,000					3,600 lf	$252,000	

DOWNSTREAM CHANNELIZATION (Simpson Wash)	$40 /sy	1,111 sy	$44,444					1,111 sy	$44,444	
DOWNSTREAM CHANNELIZATION (18th Street Wash)	$40 /sy	4,000 sy	$160,000					4,000 sy	$160,000	
3-8'x 8'x 46' RCBC (Mission Lane)	$350 /cy	124 cy	$43,470					124 cy	$43,470	
DRAINAGE EASEMENTS (Simpson Wash)	$5 /sf	18,000 sf	$90,000					18,000 sf	$90,000	
DRAINAGE EASEMENTS (18th Street Wash)	$5 /sf	30,000 sf	$150,000					30,000 sf	$150,000	
UTILITY RELOCATION				**$1,000,000**			**$200,000**			**$1,000,000**
FOR CROSS DRAINAGE STRUCTURES	$500,000 /ls	1 ls	$500,000					1 ls	$500,000	
I10 CROSSINGS	$500,000 /ls	1 ls	$500,000		1 ls	$200,000		1 ls	$500,000	
MISCELLANEOUS				**$1,446,000**			**$946,000**			**$1,446,000**
CONSTRUCT & REMOVE DETOURS	$12 /sy	8,000 sy	$96,000		8,000 sy	$96,000		8,000 sy	$96,000	
SIGNING & STRIPING		1 ls	$600,000		1 ls	$350,000		1 ls	$600,000	
LANDSCAPING		1 ls	$750,000		1 ls	$500,000		1 ls	$750,000	
LIGHTING & SIGNALS				**$710,000**			**$215,000**			**$710,000**
RELOCATE HIGH MAST POLES, FOUNDATION, CONDUIT	$15,000 /ea	34 ea	$510,000		1 ea	$15,000		34 ea	$510,000	
TRAFFIC SIGNALS (Congress St)	$100,000 /ea	2 ea	$200,000		2 ea	$200,000		2 ea	$200,000	
	SUBTOTAL		**$49,721,131**	**$49,721,131**		**$17,243,257**	**$17,243,257**		**$50,906,298**	**$50,906,298**
TRAFFIC CONTROL @ 5%			**$2,474,057**			**$862,163**			**$2,533,315**	
CONTINGENCIES @ 15%			**$7,458,170**			**$2,586,489**			**$7,635,945**	
CONSTRUCTION ADMINISTRATION @ 15%			**$7,458,170**			**$2,586,489**			**$7,635,945**	
DESIGN @ 15%			**$7,458,170**			**$2,586,489**			**$7,635,945**	
	TOTAL		**$74,569,697**			**$25,864,886**			**$76,347,448**	
ABUTMENT FOR FUTURE BRIDGE FROM PEDESTRIAN CROSSING TO CLARK	$3,200 /lf	600 lf	$1,920,000					600 lf	$1,920,000	
	TOTAL		$76,489,697			$25,864,886			$78,267,448	

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